KANSAS CITYS, MISSOURI AND KANSAS FLOOD RISK MANAGEMENT PROJECT

COMMUNICATION

FROM

THE ASSISTANT SECRETARY OF THE ARMY, CIVIL WORKS, DEPARTMENT OF DEFENSE

TRANSMITTING

THE KANSAS CITYS, MISSOURI AND KANSAS FLOOD RISK MANAGEMENT PROJECT REPORT FOR MAY 2014

PART 1 of 2

MAY 23, 2016.—Referred to the Committee on Transportation and Infrastructure and ordered to be printed

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DEPARTMENT OF THE ARMY
OFFICE OF THE ASSISTANT SECRETARY
CIVIL WORKS
108 ARMY PENTAGON
WASHINGTON DC 20310-0108

MAR 28 2016

Honorable Paul Ryan
Speaker of the House of Representatives
U.S. Capitol Building, Room H-232
Washington, DC 20515

Dear Mr. Speaker:

In response to a study conducted under the authority of section 216 of the Flood Control Act of 1970, the Secretary of the Army supports the authorization and construction of the Armourdale and Central Industrial District Levee Units at Kansas Citys, Missouri and Kansas for the purpose of flood risk management. The proposal is described in the report of the Chief of Engineers, dated January 27, 2015, which includes other pertinent documents. The Secretary of the Army plans to implement the project at the appropriate time, considering National priorities and the availability of funds.

The study has been conducted in two phases. Phase 1 resulted in an Interim Feasibility Study and Environmental Impact Statement published in 2006 presenting study results of the entire levee system and recommendations for the Argentine, Fairfax-Jersey Creek, North Kansas City, East Bottoms and Birmingham Levee Units of the system. This final feasibility study completes Phase 2 of the study and presents recommendations for the Armourdale and Central Industrial District Units of the system. The recommended plan is the Locally Preferred Plan (LPP) that is smaller scale and lower cost than the National Economic Development (NED) plan. The LPP qualifies as a Categorical Exemption to the NED plan, as defined in ER 1105-2-100, because net benefits continue to increase as the non-Federal constraint of not exceeding phase I performance levels are reached. The plan includes measures to increase the performance of the existing Armourdale and Central Industrial District Levee Units by addressing structural and geotechnical reliability of existing features, and increasing the height of the existing levees and floodwalls by as much as five additional feet.

The plan for the Central Industrial District Levee Unit on the Kansas River include increasing the height of approximately 11,750 linear feet of levee and floodwall between 0.2 and 3.8 feet (average increase 3.6 feet), adding 600 linear feet of new floodwall, adding underseepage control features including 57 relief wells and approximately 3,450 linear feet of area fill, adding four new closure structures and modifying or replacing two closures, modifying five pump stations and removing two stations, modifying drainage structures, and relocating utility crossings. The plan for the Central Industrial District Levee Unit on the Missouri River includes modifying approximately 290 linear feet of floodwall to improve structural reliability. The plan for the Armourdale Levee Unit on the Kansas River includes increasing the height of approximately 33,000 linear feet of levee and floodwall between 1.2 and 5.2 feet (average increase 4 feet), adding underseepage control measures including 74 relief wells and 2,000 linear feet of underground slurry cutoff wall, adding three closure structures and modifying or replacing four closures, modifying seven pump stations and removing two stations, modifying drainage structures, and relocating utility crossings.

Based on a discount rate of 3.125 percent and a 50-year period of economic analysis, the total equivalent average annual costs of the proposed project are estimated to be $15,997,100, including monitoring and operations, maintenance, repair, rehabilitation, and replacement. The recommended plan has total annual benefits of $58,975,600 with net annual benefits of $42,978,600. The benefit-to-cost ratio is 3.7 to 1.

A Record of Decision was prepared in accordance with the National Environmental Policy Act reaffirming the analysis in the Phase I environmental impact statement dated August 2006 applies to the Phase II recommended plan. The recommended plan has been identified as the environmentally preferred plan. Adverse environmental impacts have been avoided and minimized where practicable. No compensatory mitigation is required.

The Independent External Peer Review was completed by Battelle Memorial Institute. The review comments resulted in expanded narratives throughout the report to support the decision-making process and justify the recommended plan. All comments from the above referenced reviews have been addressed and incorporated into the final documents.

The Office of Management and Budget (OMB) advises that there is no objection to the submission of the report to Congress and concludes that the report recommendation is consistent with the policy and programs of the President. However, OMB also noted that the project would need to compete with other proposed investments for funding in future budgets. A copy of OMB's letter, dated March 11, 2016, is enclosed. I am providing a copy of this transmittal and the OMB letter to the Subcommittee on Water Resources and Environment of the House Committee on Transportation and Infrastructure, and the Subcommittee on Energy and Water Development of the House Committee on Appropriations. I am also providing an identical letter to the President of the Senate.

Very truly yours,

Jo-Ellen Darcy

Jo-Ellen Darcy
Assistant Secretary of the Army
(Civil Works)

Enclosures

5 Enclosures

1. OMB Clearance letter, dated March 11, 2016
2. Record of Decision, dated March 28, 2016
3. Chief's Report, January 21, 2015
4. State and Agency review letters
5. Final Feasibility Report, May 2014, Kansas Citys, Missouri and Kansas Flood Risk Management Study (CD)

EXECUTIVE OFFICE OF THE PRESIDENT
OFFICE OF MANAGEMENT AND BUDGET
WASHINGTON, D.C. 20503

March 11, 2016

The Honorable Jo-Ellen Darcy
Assistant Secretary of the Army (Civil Works)
108 Army Pentagon
Washington, DC 20310-0108

Dear Ms. Darcy:

As required by Executive Order 12322, the Office of Management and Budget has reviewed a May 2014 Army Corps of Engineers final feasibility report recommending the Armourdale and Central Industrial District Units, Missouri River and Tributaries at Kansas Citys (Phase 2) project.

Based on an analysis of the project's costs and benefits, the Corps estimated that the benefit-cost ratio for the project is 3.5 to 1 at a discount rate of 3.375 percent, which is the discount rate that the Corps is required to use for FY 2015 under section 80 of the Water Resources Development Act of 1974 to evaluate and formulate its proposed water resources projects. According to the Corps, the equivalent benefit-cost ratio is 1.6 to 1 at a seven percent discount rate. This is the discount rate that the Administration uses in the Budget to measure the performance of Corps construction projects whose primary purpose is to provide an economic return to the Nation.

The report recommends improvements to two levee units (Armourdale and Central Industrial District). However, the effects of these improvements are linked hydrologically to work on an adjacent, third levee unit (Argentine), which was authorized in the Water Resources Development Act of 2007. For investment purposes, we believe that the economic return for these three levee units should be evaluated accordingly, as an investment in a single, integrated unit consisting of the work that the Corps has recommended for these three levees.

The Office of Management and Budget does not object to you submitting this report to the Congress. When you do so, please advise the Congress that an authorization to construct this project would be consistent with the programs and policies of the President. Further, should the Congress authorize this project for construction, it would need to compete with other proposed investments for funding in future budgets.

Sincerely,

John Pasquantino
Deputy Associate Director
Energy, Science, and Water

RECORD OF DECISION

KANSAS CITYS, MISSOURI AND KANSAS FLOOD RISK MANAGEMENT PROJECT

The Final Feasibility Report for Kansas Citys, Missouri and Kansas Flood Risk Management Project (Phase II), dated May 2014, addresses flood risk management problems for the second phase of the Interim Feasibility Report and Environmental Impact Statement (IFR/EIS) (Phase I), dated August 2006 with addendum dated December 2006. The Phase I report and EIS analyzed alternatives and environmental impacts for the seven levee units that compromise the Kansas Citys Local Flood Risk Management Project including the Argentine, East Bottoms, Fairfax-Jersey Creek, Birmingham, North Kansas City, Armourdale and Central Industrial District (CID) levee units. The Phase I report identified a recommended plan for five of the seven levee units: the Argentine, Fairfax-Jersey Creek, North Kansas City, East Bottoms and Birmingham levee units. The Phase I report resulted in a report of the Chief of Engineer's (Chief's Report), dated December 19, 2006, and a Record of Decision (ROD) dated November 21, 2007. The impacts to the environment from potential alternatives at Armourdale and CID levee units, including the ultimately recommended plan for these segments, were analyzed in the Phase I IFR/EIS. However, the Phase I ROD only addressed the decision for the Argentine, Fairfax-Jersey Creek, North Kansas City, East Bottoms and Birmingham levee units.

The Phase II feasibility report and the Phase I IFR/EIS addresses the effects of the flood risk management improvements on the natural system and the human environment for the Armourdale and CID levee units. The final recommendation is contained in the Chief's Report, dated January 27, 2015. The Phase II feasibility report, Phase I IFR/EIS, and the Chief's Report are incorporated herein by reference. Based on these reports, the reviews of other Federal, State and local agencies, input from the public, and review by my staff, I find the plan recommended by the Chief of Engineers to be technically feasible, economically justified, in accordance with environmental statutes, and in the public interest.

The recommended plan is the Locally Preferred Plan and includes:

- Improvements to the CID levee unit on the Kansas River:
  - Increasing the levee unit height of approximately 11,750 linear feet of levee and floodwall between 0.2 and 3.8 feet (average increase 3.6 feet);
  - Constructing an additional 600 linear feet of new floodwall;
  - Installing underseepage control features including 57 relief wells and approximately 3,450 linear feet of area fill;
  - Installing four new closure structures and modifying or replacing two closures to the CID levee unit,
  - Modifying five pump stations and removing two stations,
  - Modifying drainage structures, and relocating utility crossings; and,
  - Modifying approximately 290 linear feet of floodwall to improve structural reliability.

- Improvements to the Armourdale Levee Unit on the Kansas River:
  - Increasing the levee unit height of approximately 33,000 linear feet of levee and floodwall between 1.2 and 5.2 feet (average increase 4 feet),
  - Installing underseepage control measures including 74 relief wells and 2,000 linear feet of underground slurry cutoff wall;
  - Installing three closure structures and modifying or replacing four closures,
  - Modifying seven pump stations and removing two stations, and,
  - Modifying drainage structures, and relocating utility crossings.

The alternatives identified, evaluated and recommended in the Phase II feasibility report are the same scope and location as addressed in the Phase I IFR/EIS. In addition to the "no action" plan, six alternatives for the CID levee unit and various alternatives for each of the seven reaches of the Armourdale levee unit were evaluated. The differences among the CID alternatives are related to a proposed new tieback measure; whether or not a new tieback is included, where the tieback connection is located along the existing alignment, the effect of the new tieback on the proposed relief well system, and the alignment of the tieback between the existing unit and the bluff. For most reaches of the Armourdale Unit, only one alternative plan was identified as technically feasible and effective to perform the raise and address the respective impacts to appurtenant structural and geotechnical features. However, multiple alternative plans for structural modifications were identified and evaluated for five reaches within the Armourdale Unit. For these reaches, the alternatives that avoid encroachments and impacts to adjacent businesses and known hazardous, toxic, and radioactive waste areas were included in the recommended plan. The non-structural measures considered included structure removal or relocation, structure elevation, and structure flood proofing. The non-structural measures were eliminated as they were determined not to be efficient or effective at managing the flood risk.

The effects of the recommended plan do not exceed the effects discussed in the Phase I IFR/EIS. Accordingly, impacts to natural resources are minor and may include the removal of some grasses, weeds and incidental seedlings. In these Armourdale and CID levee units, impacts to mature trees and wetlands are not anticipated. All practicable means to avoid and/or minimize adverse environmental effects have been incorporated into the recommended plan. No compensatory mitigation is required. The recommended plan is the environmentally preferred alternative.

In addition to the public review process conducted for the Phase I IFR/EIS, the public review was conducted on the Phase II feasibility report. The public review of the draft Phase II feasibility report was completed on December 21, 2013. All comments from the public are addressed in the final Phase II feasibility report. The state and agency review of the final Phase II feasibility report and draft Chief's Report was completed June 22, 2014. Comments from state and Federal agencies did not result in any changes to the final Phase II feasibility report.

The combination of recommendations from the Phase II feasibility report and the Phase I IFR/EIS represent a complete and complimentary effort that addresses the existing Kansas Citys Flood Risk Management System as a whole. The two phases of the study effort have maintained a consistent approach to improving performance and reliability within the system. Although the Phase I IFR/EIS recommendations have previously been authorized, it is important to recognize the overarching systems approach to metropolitan flood risk management by evaluating the two sets of recommendations together and presenting a total system recommendation. When Congress authorized Phase I of the project pursuant to section 1001 of the Water Resources Development Act of 2007 (Public Law 110-114), it was understood that Phase I was a partial response to addressing the flood risk management problems in the system. The Phase II recommended plan is necessary to more completely provide for system wide flood risk management benefits.

Technical, environmental, economic and cost-effective criteria used in the formulation of alternative plans were those specified in the Water Resource Council's 1983 Economic and Environmental Principles and Guidelines for Water and Related Land Resource Implementation Studies. All applicable laws, executive orders, regulations, and guidelines were considered in evaluating alternatives. Based on review of these evaluations, I find that the recommended plan benefits outweigh the costs and any adverse effects. This Record of Decision completes the National Environmental Policy Act process.

March 28, 2016
Date

Jo-Ellen Darcy
Jo-Ellen Darcy
Assistant Secretary of the Army
(Civil Works)

DEPARTMENT OF THE ARMY
CHIEF OF ENGINEERS
2600 ARMY PENTAGON
WASHINGTON, DC 20310-2600

REPLY TO
ATTENTION OF

1 7 JAN 2015

DAEN

SUBJECT: Armourdale and Central Industrial District Levee Units, Missouri River and Tributaries at Kansas Citys, Missouri and Kansas

THE SECRETARY OF THE ARMY

1. I submit for transmission to Congress my report on proposed modifications to the Armourdale and Central Industrial District levee units of the Missouri River and Tributaries at Kansas Citys, Missouri and Kansas, project. It is accompanied by the report of the Kansas City District Engineer and the Northwestern Division Engineer, which address modifying the project authority to improve project capabilities and reliability. These reports were prepared under the authority of Section 216 of the 1970 Flood Control Act, which authorizes the Secretary of the Army to review the operation of projects constructed by the Corps of Engineers when found advisable due to significantly changed physical, economic or environmental conditions. The Missouri River and Tributaries at Kansas Citys project is authorized by the Flood Control Act of 1936, and modified by the Flood Control Acts 1944, 1946, 1954, and 1962, and the Water Resources Development Act of 2007. Preconstruction engineering and design activities, if funded, would be continued under the Section 216 authority.

2. The reporting officers recommend authorization of a plan for flood risk management to modify the existing project to reduce flood risks in the vicinity of Kansas City, Missouri, and Kansas City, Kansas. The plan includes measures to increase the performance of the existing Armourdale and Central Industrial District Levee Units, which are part of the existing Kansas Citys system. The increase in performance is achieved by addressing structural and geotechnical reliability of existing features, and increasing the height of the existing levees and floodwalls by as much as five additional feet. The recommended plan provides approximately 65% assurance to contain flows within the project parameters at or below 0.2% (1/500) Annual Exceedance Probability (AEP) water surface elevation, consistent with the existing flood risk management system. This is the equivalent of the recommended plan providing approximately 98% assurance to contain flows within the project parameters at or below the 1.0% (1/100) AEP water surface elevation.

3. The recommended plan would reduce flood risk to areas of the Citys of Kansas City, Missouri, and Kansas City, Kansas. The proposed plan would reduce Expected Annual Damages (EAD) by 88%, with a residual EAD of approximately $7.7M. Annual Exceedance Probabilities for flooding from the Kansas River would be reduced from 3.5% in the Armourdale Unit and 0.33% in the Central Industrial District Unit to 0.12% in both units. The proposed project was evaluated in the 2006 Programmatic Environmental Impact Statement. No significant changes were identified and the determination that no long-term effect on environmental resources was confirmed. No compensatory mitigation is required.

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4. Based on October 2014 price levels, the total first cost of these measures is estimated at $318,517,000 for all flood risk management. Under cost sharing specified by Section 103 of the Water Resources Development Act (WRDA) of 1986, Public Law 99-662, as amended by Section 202 of WRDA 1996, each measure would be cost shared 65 percent federal and 35 percent non-federal, resulting in an estimated federal share of $207,036,000 and an estimated non-federal share of $111,481,000. The total expected annual costs, based on a discount rate of 3.375 percent and a 50-year period of analysis, are $16,876,900, including $347,900 for operation, maintenance, repair, rehabilitation, and replacement (OMRR&R). The expected annual benefits are estimated to be $57,565,300 with net annual benefits of $40,688,400. The benefit-cost ratio is approximately 3.4 to 1 for the new work. The measures recommended for implementation will be carried out with two non-federal cost sharing sponsors.

a. The recommended measures for increasing the degree of protection for the Armourdale Levee Unit on the Kansas River include increasing the height of approximately 33,000 linear feet of levee and floodwall between 1.2 and 5.2 feet (average increase 4 feet), adding underseepage control measures including 74 relief wells and 2,000 linear feet of underground slurry cutoff wall, adding three closure structures and modifying or replacing four closures, modifying seven pump stations and removing two stations, modifying drainage structures, and relocating utility crossings. The Kaw Valley Drainage District is the non-federal cost-sharing sponsor for all features. The estimated total first cost of the plan is $236,447,000. The estimated federal share is $153,690,500 and the estimated non-federal share is $82,756,500. The cost of lands, easements, rights-of-way, relocations, and dredged or excavated material disposal areas (LERRD) is estimated at $4,532,000. There is no cost associated with mitigation due to the low potential to impact the existing environment in and around the project site. The total expected annual costs are $12,183,900, including $198,200 for OMRR&R. The expected annual benefits are estimated to be $52,254,600 with net annual benefits of $40,070,700.

b. The recommended measures for increasing the degree of protection for the Central Industrial District Levee Unit on the Kansas River include increasing the height of approximately 11,750 linear feet of levee and floodwall between 0.2 and 3.8 feet (average increase 3.6 feet), adding 600 linear feet of new floodwall, adding underseepage control features including 57 relief wells and approximately 3,450 linear feet of area fill, adding four new closure structures and modifying or replacing two closures, modifying five pump stations and removing two stations, modifying drainage structures, and relocating utility crossings,. The Kaw Valley Drainage District is the non-federal cost-sharing sponsor for all features. The estimated total first cost of the plan is $81,485,000. The estimated federal share is $52,965,300 and the estimated non-federal share is $28,519,700. The cost of lands, easements, rights-of-way, relocations, and dredged or excavated material disposal areas (LERRD) is estimated at $2,631,000. There is no cost associated with mitigation due to the low potential to impact the existing environment in and around the project site. The total expected annual costs are $4,292,600, including $149,700 for OMRR&R. The expected annual benefits are estimated to be $5,246,900 with net annual benefits of $954,300.

c. The recommended measures for increasing the degree of protection for the Central Industrial District Levee Unit on the Missouri River includes modifying approximately 290 linear feet of floodwall to improve structural reliability. The City of Kansas City, Missouri, is the non-federal

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cost-sharing sponsor for all features. The estimated total first cost of the plan is $585,000. The estimated federal share is $380,300 and the estimated non-federal share is $204,700. The cost of lands, easements, rights-of-way, relocations, and dredged or excavated material disposal areas (LERRD) is estimated at $0. There is no cost associated with mitigation due to the low potential to impact the existing environment in and around the project site. The total expected annual costs are $29,500, including $0 for OMRR&R. The expected annual benefits are estimated to be $63,600 with net annual benefits of $34,000.

5. The above plan for increasing the degree of protection and benefit for the Armourdale and Central Industrial District Units complete the total system evaluation and recommendation for improving the benefits provided by the existing Kansas Citys Flood Risk Management Project. The previously approved plan for modifications to this system is currently being implemented.

a. The plan to increase the degree of protection for the Argentine Levee Unit and to improve the reliability of the East Bottoms and Fairfax-Jersey Creek Levee Units were previously recommended by the Chief's Report of Dec. 19, 2006, and authorized in the Water Resources Development Act (WRDA) of 2007. Based on October 2014 price levels the authorized total first cost of these three measures is estimated at $81,514,000, all for flood risk management. Under cost sharing specified by Section 103 of the WRDA of 1986, Public Law 99-662, as amended by Section 202 of WRDA 1996, the estimated federal share is $52,984,100 and the estimated non-federal share is $28,529,900.

b. The plan to correct design and construction deficiencies in the Fairfax-Jersey Creek and North Kansas City levee units in order to restore the original degree of protection were approved by the Chief's Report of Dec 19, 2006. Based on October 2014 price levels, the authorized total first cost of the deficiency correction plan is estimated at $20,700,000. In accordance with Section 103 of WRDA 1986, as amended, the estimated federal share is $13,455,000 and the estimated non-federal cost share is $7,245,000.

6. The goals and objectives included in the Campaign Plan of the U.S. Army Corps of Engineers have been fully integrated into the feasibility study process. The recommended plan has been designed to avoid or minimize environmental impacts, to reduce risk of loss of life, and to reasonably maximize economic benefits to the community in coordination with the existing flood risk management system. The Feasibility Study team organized and participated in stakeholder and public meeting throughout the process and worked to achieve a balance of project goals and public concerns. The study report fully describes local flood risks associated with the Kansas River, including residual risks that remain even after implementation of the recommended plan. These residual risks have been communicated to the non-federal sponsors and they understand and agree with the analysis. The Feasibility Study team has reviewed current available information on the estimated future impact of climate change in the region. While a trend towards wetter conditions in the future has been identified, the impacts are expected to be within the range of uncertainty addressed by the current hydrologic model.

7. In accordance with the Corps guidance on review of decision documents, all technical, engineering and scientific work underwent an open, dynamic and rigorous review process to ensure

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technical quality. This included an Agency Technical Review (ATR), and an Independent External Peer Review (Type I IEPR), and a Corps Headquarters policy and legal review. All concerns of the ATR have been addressed and incorporated into the final report. An IEPR was completed by Battelle Memorial Institute in January 2014. Overall, the IEPR report contained twenty-one comments from two commenting periods. The first comment period was conducted at the Alternative Formulation Briefing (AFB) and the second round of comments was on the draft final feasibility report. Five comments of high significance were identified at the AFB and one comment of high significance was identified within the draft final feasibility report. The IEPR comments identified concerns in areas of the engineering assumptions and environmental analysis that needed improvements to support the decision-making process and plan selection. This resulted in expanded narratives throughout the report to support the decision-making process and to justify the recommended plan. All comments from the above referenced reviews have been addressed and incorporated into the final document. Overall the reviews resulted in improvements to technical quality of the report. A safety assurance review (Type II IEPR) will be conducted during the design phase of the project.

8. Washington level review indicated that the plan recommended by the reporting officers is technically sound, economically justified, and environmentally and socially acceptable. The plan complies with the essential elements of the 1983 U.S. Water Resources Council's Economic and Environmental Principles and Guidelines for Water and Land Related Resources Implementation Studies and complies with other administrative and legislative policies and guidelines. The views of interested parties, including federal, State, and local agencies have been considered during the State and Agency review period. During this review USEPA requested additional information regarding the potential impacts of future regional climate changes on the projects performance and the integration of non-structural measures. In response to these concerns USEPA was provided analysis that shows that there is little effect to project performance due to regional climate change. Non-structural measures were considered in this study, however; those measures were determined not to be cost effective.

9. I concur with the findings, conclusions and recommendation of the reporting officers. Accordingly, I recommend the plan to further reduce flood risks for the Missouri River and Tributaries at Kansas Citys project be authorized at an estimated total first cost of $318,517,000 with such modifications as in the discretion of the Chief of Engineers may be advisable. My recommendation and approval are subject to cost sharing, financing, and other applicable requirements of federal and state laws and policies, including Section 103 of WRDA 1986, as amended. The non-federal sponsors would provide the non-federal cost share and all LERRD. Further, the non-federal sponsors would be responsible for all OMRR&R. This recommendation and approval are subject to the non-federal sponsors agreeing to comply with all applicable federal laws and policies, including but not limited to:

a. Provide the non-federal share of total project costs, including a minimum of 35 percent but not to exceed 50 percent of total project costs as further specified below:

(1) Provide 35 percent of design costs in accordance with the terms of a design agreement entered into prior to commencement of design work for the project;

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(2) Provide, during construction, a contribution of funds equal to 5 percent of total project costs;

(3) Provide all lands, easements, and rights-of-way, including those required for relocations, the borrowing of material, and the disposal of dredged or excavated material; perform or ensure the performance of all relocations; and construct all improvements required on lands, easements, and rights-of-way to enable the disposal of dredged or excavated material all as determined by the government to be required or to be necessary for the construction, operation, and maintenance of the project; and

(4) Provide, during construction, any additional funds necessary to make its total contribution equal to at least 35 percent of total project costs.

b. Not less than once each year, inform affected interests of the extent of protection afforded by the project.

c. Agree to participate in and comply with applicable federal floodplain management and flood insurance programs.

d. Comply with Section 402 of the Water Resources Development Act of 1986, as amended (33 U.S.C. 701b-12), which requires a non-federal interest to prepare a floodplain management plan within one year after the date of signing a project partnership agreement, and to implement such plan not later than one year after completion of construction of the project.

e. Publicize floodplain information in the area concerned and provide this information to zoning and other regulatory agencies for their use in adopting regulations, or taking other actions, to prevent unwise future development and to ensure compatibility with protection levels provided by the project.

f. Prevent obstructions or encroachments on the project (including prescribing and enforcing regulations to prevent such obstructions or encroachments) such as any new developments on project lands, easements, and rights-of-way or the addition of facilities which might reduce the level of protection the project affords, hinder operation and maintenance of the project, or interfere with the project's proper function.

g. For so long as the project remains authorized, operate, maintain, repair, rehabilitate, and replace the project, or functional portions of the project, including any mitigation features, at no cost to the federal government, in a manner compatible with the project's authorized purposes and in accordance with applicable federal and State laws and regulations and any specific directions prescribed by the federal government.

h. Hold and save the United States free from all damages arising from the construction, OMRR&R of the project and any betterments, except for damages due to the fault or negligence of the United States or its contractors.

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i. Perform, or ensure performance of, any investigations for hazardous substances that are determined necessary to identify the existence and extent of any hazardous substances regulated under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), Public Law 96-510, as amended (42 U.S.C. 9601-9675), that may exist in, on, or under lands, easements, or rights-of-way that the federal government determines to be required for construction, operation, and maintenance of the project. However, for lands that the federal government determines to be subject to the navigation servitude, only the federal government shall perform such investigations unless the federal government provides the non-federal sponsor with prior specific written direction, in which case the non-federal sponsor shall perform such investigations in accordance with such written direction.

j. Assume, as between the federal government and the non-federal sponsor, complete financial responsibility for all necessary cleanup and response costs of any hazardous substances regulated under CERCLA that are located in, on, or under lands, easements, or rights-of-way that the federal government determines to be required for construction, operation, and maintenance of the project.

k. Agree, as between the federal government and the non-federal sponsor, that the non-federal sponsor shall be considered the operator of the project for the purpose of CERCLA liability, and to the maximum extent practicable, operate, maintain, repair, rehabilitate, and replace the project in a manner that will not cause liability to arise under CERCLA.

10. The recommendation contained herein reflects the information available at this time and current departmental policies governing formulation of individual projects. It does not reflect program and budgeting priorities inherent in the formulation of a national civil works construction program nor the perspective of higher review levels within the executive branch. Consequently, the recommendation may be modified before it is transmitted to Congress as a proposal for authorization and implementation funding. However, prior to transmittal to Congress, the sponsors, the States of Kansas and Missouri, interested federal agencies, and other parties will be advised of any modifications and will be afforded the opportunity to comment further.

THOMAS P. BOSTICK
Lieutenant General, USA
Chief of Engineers

From: Kramer, Mark J HQ
To: Abadie, William D NWP
Subject: FW: Kansas City Levees Project (UNCLASSIFIED)
Date: Thursday, September 04, 2014 9:57:14 AM

Classification: UNCLASSIFIED
Caveats: NONE

-----Original Message-----
From: Bee, Patricia L HQ02
Sent: Thursday, July 03, 2014 4:17 PM
To: Nicholson, Scott R HQ02; Kramer, Mark J HQ
Subject: FW: Kansas City Levees Project (UNCLASSIFIED)

Classification: UNCLASSIFIED
Caveats: NONE

EPA response.

-----Original Message-----
From: Shepard, Larry [mailto:Shepard.Larry@epa.gov]
Sent: Thursday, July 03, 2014 3:56 PM
To: Bee, Patricia L HQ02
Subject: [EXTERNAL] RE: Kansas City Levees Project (UNCLASSIFIED)

I have reviewed my comments on the draft Final Feasibility Report for the Kansas Citys, Missouri and Kansas, and scanned the Final Feasibility Report for its final treatment of several selected topics identified in those earlier comments.

With regard to my earlier comments regarding the treatment of the potential impacts of future regional climate changes affecting the performance of flood risk reduction measures in the project area, the FFR simply provides broad characterization of potential future changes to hydrology and dismisses any possible changes in risk resulting from climate change as being within "the bands of uncertainty in the Existing Condition Feasibility hydrologic analysis." Comments offered in regard to the integration of non-structural measures with structural measures in the wider geographic context of the flood risk reduction for the metropolitan area were similarly dismissed based on those approaches being generally unsuitable or as not addressing any of the objectives specific to the existing system.

I have no further comments with regard to the Final Feasibility Report. Thank you for the opportunity to review the final report.

Larry Shepard
NEPA Team
U.S. Environmental Protection Agency
Region 7
11201 Renner Blvd.
Lenexa, Kansas 66219
913-551-7441
shepard.larry@epa.gov

Classification: UNCLASSIFIED
Caveats: NONE

Classification: UNCLASSIFIED

Agency	Nature of Contact/Date	Comment Summary
Missouri Department of Conservation	Submitted via letter dated 12/20/2013	Concerned with potential increase in downstream flood impacts due to levee raise.
Response Summary		
At their existing heights the Armourdale and CID Kansas Levee units are currently able to pass the 0.5% (200 year) event. As noted in the Phase 2 report there is a proposed raise for the Armourdale and CID Levee units to an elevation equivalent to the 0.2% (500 year) event plus 3 feet (500 year +3). Under existing conditions the Armourdale and CID-Kansas levee units would be overtopped by an average of 0.6 feet and 1.1 feet respectively during the 500 year event (341,000 cfs) primarily along the upstream segments of each levee. However, levee weir flow calculations show that the CID-Kansas levee unit would have a maximum overflow into the protected area of 3,800 cfs, which is only 1.1% of the total 500 year flow. In addition, weir flow calculations for the Armourdale levee unit show that the protected area would be inundated and filled prior to the 500 year peak overtopping elevation of I.I feet. As such any water overtopping the upstream portion of the levee would be returned to the Kansas River on the downstream end of the levee as the maximum flow and depth is approached. This would cancel out any flow reduction due to overtopping at the upstream end of the levee. Based on this evaluation, raising the Armourdale and CID-Kansas Levee units would cause approximately 3,800 cfs in additional flow at the Kansas/Missouri River confluence for the 500 year event. This equates to a 0.7% increase in peak flow and a 1.3 inch increase in water surface elevation immediately downstream of the Kansas River / Missouri River confluence. These increases are considered to be negligible. The overtopping weir flow calculation depths and durations were based on the rate of rise observed during the 1993 flood event (1.7 feet/day)."		

Agency	Nature of Contact/Date	Comment Summary
U.S. Environmental Protection Agency	Submitted vie e-mail dated 12/31/13	1. Draft Report Section II.A. The document would be improved with a characterization of the relationship between unit structures' "design discharges" and flood "design frequency" as they affect structure height, i.e., design discharge of 390,000 cfs and levee flood profile at flood frequency of 0.2%
Response Summary		
The Discharge-Frequency relationships summarized in Table 3-10 of the Final Feasibility Report indicate that the authorized design discharge of 390,000 cfs has an annual chance exceedance of less than 0.1%. Given the very low chance of occurrence of a flood of this magnitude it was determined to be neither practical nor desired to evaluate existing performance or develop alternatives to modify the existing project for the design discharge. For consistency with the desired benefits and uniformity of risk management within the levee system, evaluations of current and future project performance conducted for this study focused on the 0.2% chance flood. This response has been added to the report text after Table 3-10.		

Comment Summary
2. Draft Report Section II.F.5.0. This section would be improved with more detail summarizing here what is provided in the 2006 EIS and the Final Feasibility Report appendices for the Armourdale and CID Units. Appendix D is thorough but the inclusion of a summary of this information in the body of the Report would improve its readability.
Response Summary
A summary of the HTRW findings from Appendix D can be found in the Final Feasibility Report Section 5.1.3.

Comment Summary
3. Draft Report Section III.B.2.0. This section does not address potential changes in precipitation patterns or intensity resulting from projected changes in regional climate change. Recognizing that the basis for future hydrological predictions is the 2003 Upper Mississippi River System Flow Frequency Study, this report should at least address the possibility that climate changes are predicted and characterize the degree to which such changes would or would not affect the performance of the planned changes to the levee system, i.e., some form of sensitivity analysis.
Response Summary
USACE guidance on climate change adaptation inputs for inland hydrology is at the draft final stage of production, and has not yet been officially released for use. As such, there was no guidance in place when the hydrologic analysis was conducted (finalized 2006) for the Kansas City Levees Feasibility Study. The proposed USACE guidance will initially recommend a qualitative approach. A summary of the qualitative approach as would be applied to the Kansas City Levees is provided below. The climate of northeast Kansas trends toward a continental weather pattern of cold winters and hot, humid summers. The average temperature in 2013 at Topeka, KS (which represents the northeast portion of Kansas) was 60 degrees. The average high temperature was 73 and average low temperature was 47. The average yearly precipitation was about 37 inches of moisture. A model of future conditions for the central plains of the United States was created by the NOAA National Environmental Satellite, Data and Information Service in a report issued in January 2013. This report is an assessment of Climate Trends and Scenarios into the next 50 to 100 years. The report cites that over the past period of record for the Kansas River basin, both temperature and precipitation has trended above normal, especially over the last 50 years. To account for climate change in the meteorological conditions, the future forecast of conditions in the region takes into consideration the past temperature and precipitation records, and then considers future modeled conditions in the area through 2070. According to the NESDIS report, a warming trend of about 3-5 degrees F and a precipitation trend very slightly toward wetter conditions can be expected through the next 50 years although significant uncertainty is expected with these estimates. Based on this slight trend toward wetter conditions frequency flows over the study basin may increase, but these increases are being treated in this evaluation to be retained within the bands of uncertainty in the Existing Condition Feasibility hydrologic analysis.

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
441 G STREET, NW
WASHINGTON, DC 20314-1000

REPLY TO
ATTENTION OF

SEP 17 2014

Planning and Policy Division

Mr. Larry Shepard
NEPA Team
U.S. Environmental Protection Agency, Region 7
11201 Renner Boulevard
Lenexa, Kansas 66219

Dear Mr. Shepard:

The purpose of this letter is to respond to your email dated July 3, 2014, regarding potential climate changes that may affect the performance of the Kansas Citys, Missouri and Kansas Flood Risk Management Project's flood risk reduction measures. The U.S. Army Corps of Engineers issued guidance, Engineering and Construction Bulletin No. 2014-10, on incorporating climate change impacts to inland hydrology in Civil Works studies. This guidance and the enclosed response should provide the necessary details to explain how risk is not likely to change as a result of the current understanding of future climate change projections.

Per Corps policy and the Floodplain Management Executive Order 11988, flood risk management projects must consider the use of non-structural measures. While we do examine the cost and benefits of such alternatives, our policies do not require us to recommend such projects. In this particular case, most of the non-structural measures (relocation, structure-elevation, flood proving, etc.) were determined not to be cost effective, efficient and/or acceptable. However, we do rely on flood-warning systems and evacuation plans as noted in the discussion on residual risk in the main report.

I hope this response adequately addresses your concerns and questions. If you have additional questions, please contact Mr. Steven A. Kopecky, Deputy Chief, Northwestern and Pacific Ocean Division Regional Integration Team, at (202) 761-4527.

Sincerely,

Theodore A. Brown

Theodore A. Brown, P.E.
Chief, Planning and Policy Division
Directorate of Civil Works

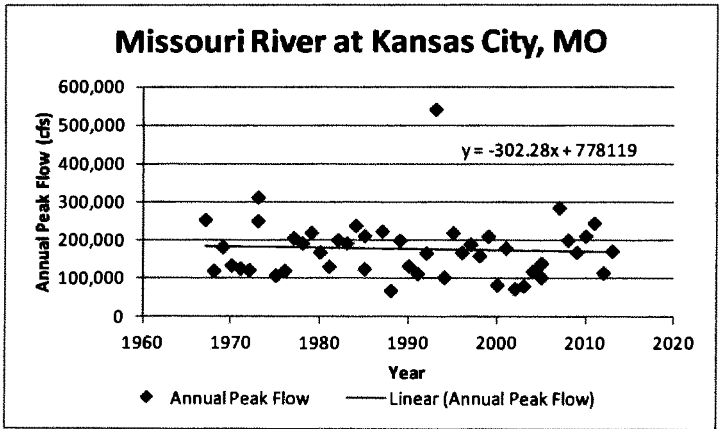
Enclosures

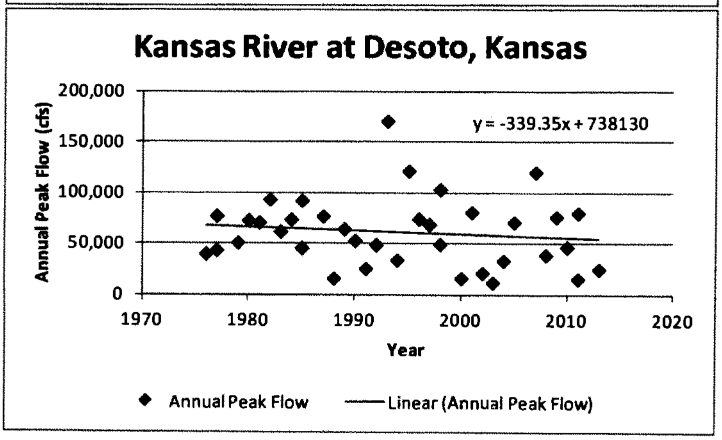
NWK RESPONSE FOR CLIMATE CHANGE

Since the time of the original comment submittal, USACE has published final guidance for incorporating climate change impacts to inland hydrology in civil works studies, designs, and projects in Engineering and Construction Bulletin (ECB) No. 2014-10 on 02 May 2014. The guidance is similar to the preliminary guidance that was reviewed to draft the initial response, however additional detail and context is provided here to better explain how risk could change with future climate change projections and trend analysis to the conclusion statement provided in the initial response.

As previously stated, a model of future conditions for the central plains of the United States was created by the NOAA National Environmental Satellite, Data and Information Service in a report issued in January 2013. This report is an assessment of Climate Trends and Scenarios into the next 50 to 100 years. The report cites that over the past period of record for the Kansas River basin, both temperature and precipitation has trended above normal, especially over the last 50 years. To account for climate change in the meteorological conditions, the future forecast of conditions in the region takes into consideration the past temperature and precipitation records, and then considers future modeled conditions in the area through 2070. According to the NESDIS report, a warming trend of about 3-5 degrees F and a precipitation trend very slightly toward wetter conditions can be expected through the next 50 years although significant uncertainty is expected with these estimates.

Trend analysis was conducted on observed annual peak flow data on the Kansas River at Desoto and Missouri River at Kansas City to help determine whether evidence exists that conditions are becoming wetter. To minimize the potential for pre-dam flows to influence the results, the period of record was checked for 1975 to 2013 for the Kansas River and 1967 to 2013 for the Missouri River. Post 2001 data has included six out of the top ten lowest annual peak flows but only two out of the top ten high flows for both rivers. Both rivers show a downward trend in annual peak streamflows over the post dam period of record on the order of 300 cfs per year when applying a linear trendline in Microsoft Excel. These results are shown in the figure below.





Additional analysis was made on the annual peak flow data sets to check for potential changes in regulated flow frequency curves in recent years. The period of record was checked for 1975 to 2001 against 1975 to 2013 for the Kansas River, and 1967 to 1997 against 1967 to 2013 for the Missouri River. Even though this is a regulated system, a Bulletin 17B analysis was computed and produced reasonable results that are useful for assessing the potential impacts of climate change. TheBulletin 17B flow frequency was compared for each time period using regional skew coefficients for the sole purpose of checking for potential change in regulated frequency curves with more recent data. This analysis showed minor changes, approximately a five percent change or smaller, in computed flows for flows larger than a 1/5 annual chance exceedance (ACE) event for both streams. Frequency flows smaller than a 1/2 ACE showed considerable downward change of 15 to 26 percent for the Kansas River, compared to a 5 to 6 percent downward shift on the Missouri River. Accordingly, the post dam annual peak flow data sets appear to be trending mostly in a downward direction, counter to projections of wetter precipitation from the NOAA modeling reports. No significant trends are present in the less frequent flow frequencies that could affect levee height design for the Kansas City Levees Project. The only potential significant changes greater than 5% change appear to be present at flows well within the channel banks on the Kansas River, more frequent than a 50 percent annual chance exceedance event.

Therefore, due to limited evidence of increasing peak flows in the trend analysis, the anticipated very slight increases from the NOAA report are being treated in this evaluation to be retained within the bands of uncertainty in the Existing Condition Feasibility hydrologic analysis.

NWK RESPONSE FOR NON-STRUCTURAL

Section 4.3.4.2 of the Final Feasibility Report was expanded to provide additional details of the types of measures, expected performance, and typical costs of implementing non-structural measures in the project study area. Ensuring that an existing levee in an urban area performs as intended and expected often does not readily provide opportunities for non-structural applications. The study area of the Kansas Citys Levees project is densely developed with residential and commercial structures at very similar elevations. The nature of flooding and potential failure of the existing project would be catastrophic in nature and overwhelm the capabilities of available flood-proofing methods of damage reduction. The cost and community impacts of relocating or raising structures to achieve the same benefit of a structural levee modification would be significant.

ENGINEERING AND CONSTRUCTION BULLETIN

US Army Corps of Engineers®

No. 2014-10 **Issuing Office:** CECW-CE **Issued:** 2 May 2014 **Expires:** 2 May 2016

Subject: Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects

Applicability: Guidance.

References: Required and related references are provided in Appendix A.

1. Purpose. This ECB provides USACE with initial guidance for incorporating climate change information in hydrologic analyses in accordance with the USACE overarching climate change adaptation policy. USACE policy requires consideration of climate change in all current and future studies to reduce vulnerabilities and enhance the resilience of our water-resource infrastructure. The guidance in this ECB is also in accordance with the President's Climate Action Plan released in June 2013 and with Executive Order 13653.

2. Objective. The objective of this ECB is to support incorporation of new science and engineering products and other relevant information about specific climate change and associated impacts in hydrologic analyses for new and existing USACE projects to enhance USACE climate preparedness and resilience.

a. This ECB is effective immediately and applies to all hydrologic analyses supporting planning and engineering decisions having an extended decision time frame. However, this guidance does not apply to operational hydrologic studies for water management or to dam safety.

b. Changes other than climate threats that affect inland hydrology will continue to be evaluated in the manner described in current USACE guidance (e.g., Chapter 18, Evaluating Change in EM 1110-2-1417, *Flood-Runoff Analysis*; and EM 1110-2-1413, *Hydrologic Analysis of Interior Areas*).

3. Introduction. USACE projects, programs, missions, and operations have generally proven to be robust enough to accommodate the range of natural climate variability over their operating life spans. Recent scientific evidence shows, however, that in some places and for some impacts relevant to USACE operations, climate change is shifting the climatological baseline about which that natural climate variability occurs, and may be changing the range of that variability as well. This is relevant to USACE because the assumptions of stationary climatic baselines and a fixed range of natural variability as captured in the historical hydrologic record may no longer be appropriate for long-term projections of the climatologic parameters, which are important in hydrologic assessments for inland watersheds. However, projections of the specific climate changes and associated impacts to local-scale project hydrology that may occur far in the future due to changing baselines and ranges of variability as reported in the recent literature are uncertain enough to require guidance on their interpretation and use. This ECB helps support the interpretation and use of climate change information for hydrologic analyses supporting planning and engineering decisions in three specific areas:

a. A qualitative assessment of potential climate change threats and impacts potentially relevant to the particular USACE hydrologic analysis being performed.

b. Resources to support the qualitative assessment of climate threats and impacts specific to those analyses.

c. An early overview of future planned guidance for additional quantitative assessments of potential climate change threats and impacts for use in future hydrologic analyses.

4. Incorporating Climate Change and Variability in Hydrologic Analyses.

a. Climate change information for hydrologic analyses includes direct changes to hydrology through changes in temperature, precipitation, and other climate variables, as well as subsequent basin responses such as sedimentation loadings potentially altered by changes in those primary climate drivers. The qualitative analysis required by this ECB includes consideration of both past (observed) changes as well as potential future (projected) changes to relevant hydrologic inputs. The results of this qualitative analysis can indicate the direction of change but not necessarily the magnitude of that change. For this reason, the qualitative analysis does not alter the numerical results of the calculations made for the other, non-climate aspects of the required hydrologic analyses. However, the climate change information synthesized and evaluated during the qualitative analysis can inform the decision process related to future without project conditions, formulation and evaluation of the performance of alternative plans, or other decisions related to project planning, engineering, operation, and maintenance.

b. The qualitative analysis is the only approach currently required for hydrologic studies for inland watersheds at the time of issuance of this ECB.

c. The qualitative analysis will be required for projects except for the following cases:

(1) Feasibility Phase: The Tentatively Selected Plan (TSP) milestone has been completed as of the date of issuance of this ECB.

(2) Preconstruction Engineering and Design (PED): The required hydrology and hydraulics components of the PED phase are more than 50% complete, as of the date of issuance of this ECB.

d. A first-order statistical analysis of the potential impacts to particular hydrologic elements of the study can be included as supplemental input to this qualitative assessment, but is not required.

e. Appendix B provides a flow chart of the guidance provided in this ECB.

f. Appendix C provides detailed guidance on how to perform the qualitative analysis, as well as an example with a first-order statistical analysis.

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5. Future Expansion of Support Documents for Implementation of this ECB. A series of guidance documents will be published in the future to support quantitative analyses of climate threats and impacts to specific project types. Appendix D provides a preview of planned future quantitative guidance.

6. HQUSACE POC. The HQUSACE POC for this action is Mr. Jerry Webb, Leader of the Hydrology, Hydraulics, and Coastal Community of Practice, 202-761-0673.

//S//
JAMES C. DALTON, P.E., SES
Chief, Engineering and Construction
U.S. Army Corps of Engineers

Encls

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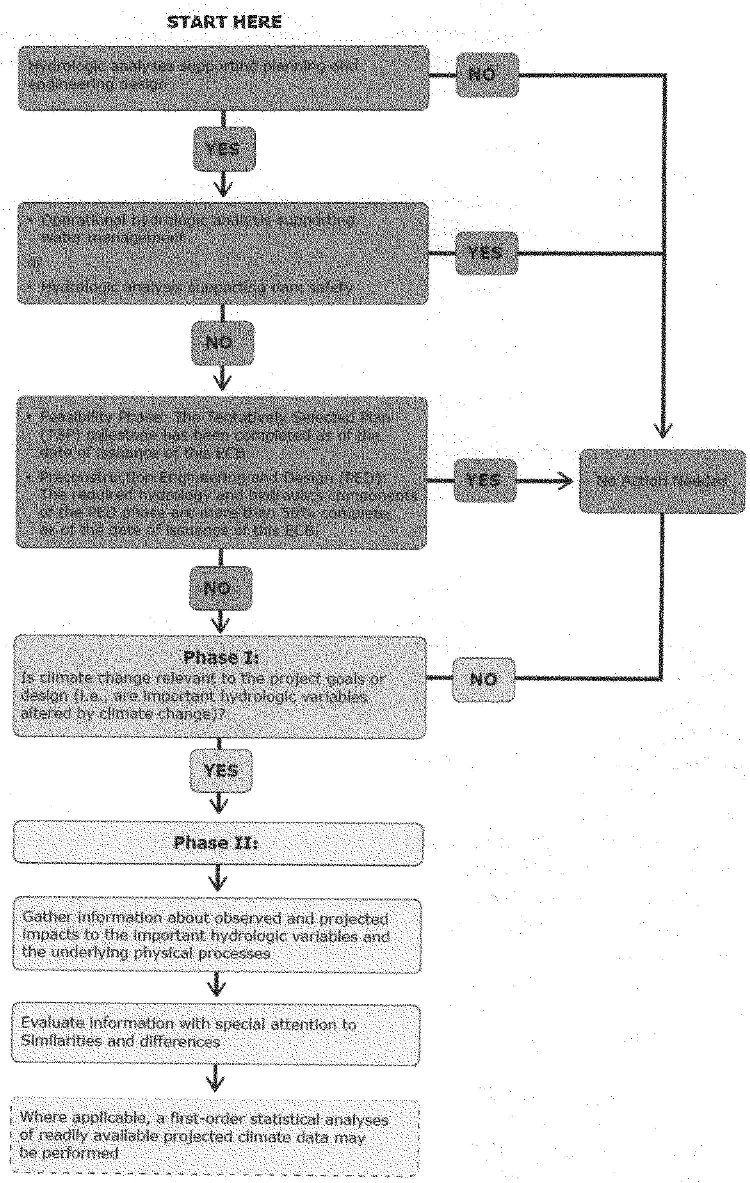
Appendix B: Flow Chart



Appendix C: Qualitative Analysis Requirements and Example.

1. Qualitative Climate Change Analysis for Hydrologic Analyses in Planning and Engineering Design Studies. The goal of a qualitative analysis of potential climate threats and impacts to USACE hydrology-related projects and operations is to describe the observed present and possible future climate threats, vulnerabilities, and impacts specific to the study goals or engineering designs. The qualitative approach on its own will not produce binding numerical outputs, but it can identify the direction of change where change is detected in climate variables relevant to elements of the hydrology study. In some cases, it may be possible to calculate an order of magnitude range of the relevant climate threats and impacts that can be considered in the context of project goals or design vulnerabilities and impacts. This, in turn, can be used to describe future without project conditions or inform decisions during the alternative formulation and selection phase, when one project alternative can be judged to reduce vulnerabilities or enhance resilience more than the others. The qualitative analysis is intended to answer a linked series of questions related to key decision components:

a. Is climate change is relevant to the project (Phase I)?

b. If yes, what is the direction of the potential climate change in the variables that may affect the hydrology of the project, and potentially impact project goals and designs (Phase II)?

2. Qualitative Analysis Framework.

a. To improve preparedness and resilience to climate change threats, USACE requires actionable science and strategies supporting informed decision-making in studies, designs, projects, and groups of projects. The certainty and applicability of the available science on climate change and hydrology that is ready for consideration in decisions varies strongly with location and spatial scale. The important consideration here is selecting information for the qualitative analysis at the appropriate scale of the study. This does not mean that the broad, global or continental-scale analyses presented with substantial expert agreement and explicit confidence estimates such as those presented in the Intergovernmental Panel on Climate Change (IPCC) synthesis documents (e.g., IPCC 2007) are not useful at the scale of USACE projects, nor that the changes in current climate and hydrologic responses observed and measured at very fine scales like those of the the Sacramento–San Joaquin [Vanrheenen et al. 2004], Upper Mississippi [Jha et al. 2006], Florida Everglades [Sklar et al. 2001], or Hudson, James, and Ungava Bays [Déry et al. 2005] cannot be used for this analysis. Rather, a successful qualitative analysis will combine the most useful information for the decisions in the hydrology study it is supporting from a range of sources, noting the differences in information types – projections and observations, e.g. – and the differences in uncertainty or confidence in the data and information deployed for the analysis.

b. The current state of actionable climate science, regardless of its scale of analysis, results in large uncertainties about projected future conditions relevant to USACE projects and programs. In some cases, these uncertainties may be comparable in scale to existing sources of uncertainty, such as future changes in land use and land cover, though the climate-related

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uncertainties can also be larger or smaller than the ones more often considered in hydrologic analyses previously. Uncertainties are different for different climate variables and in different locations and these differences should be noted in the qualitative assessment. But the climate uncertainties must be put into context with the other uncertainties relevant to the hydrologic analysis.

c. The framework of the qualitative analysis has two phases:

(1) Phase I. An initial screening-level qualitative analysis will be completed to identify whether climate change is relevant to the project goals or design in accordance with SMART Planning (i.e., are important hydrologic variables altered by climate change).

(2) Phase II. If climate change is relevant to the project goals or designs, an evaluation is made of information gathered about impacts to the important hydrologic variables and the underlying physical processes such as changes in processes governing rainfall runoff or snowmelt. The information should be used to help identify opportunities to reduce potential vulnerabilities and increase resilience as a part of the project's authorized operations and also identify any caveats or particular issues associated with the data (e.g., different literature sources may project different outcomes). The information gathered in Phase II can be included either in risk registers or separately in a manner consistent with risk characterization in planning and design studies, depending on the project phase.

3. Information Included in Phase II Qualitative Analysis. Information to support the qualitative assessment will be compiled from available, established, and reputable, scientific and engineering research literature. Where non-peer-reviewed literature is used, the assessment must include justification for its use and its peer-review equivalence. Examples of sources of peer-reviewed information on which the qualitative analyses can draw include the West-Wide Climate Risk Assessments and Basin-Wide Studies prepared by the Bureau of Reclamation (see http://www.usbr.gov/WaterSMART/wcra/), the relevant regional and sector information in the US Global Research Program's Third National Climate Assessment (see http://www.globalchange.gov/what-we-do/assessment) and subsequent updates, reports prepared for USACE climate change adaptation pilots, and reputable and peer-reviewed journal papers describing regional climate impacts to water resources. Regional synthesis information on either observations of change or projections of future change can be supplemented by additional information as described below where available.

a. Regional and Watershed Synthesis Information.

(1) Regionalized scenarios of possible future climate, as well as historic trends, are available in the National Oceanic and Atmospheric Administration (NOAA)'s Technical Report NESDIS 142, *Regional Climate Trends and Scenarios for the U.S. National Climate Assessment* (NOAA 2013). The report has sections for eight regions of the U.S., including for Alaska and for the Pacific Islands, and a ninth section for the contiguous U.S. as a whole.

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(2) Regional and sector-specific information for the United States can be obtained from the United States Global Change Research Program (USGCRP, www.globalchange.gov) and specifically the Third National Climate Assessment (NCA) released in 2014 (http://ncadac.globalchange.gov), as well as the various technical support documents to the National Climate Assessment (http://www.globalchange.gov/what-we-do/assessment/nca-activities/available-technical-inputs).

(3) Regional synthesis information for the western United States can be obtained from the Department of the Interior, Bureau of Reclamation's *Literature Synthesis on Climate Change Implications for Water and Environmental Resources* (Bureau of Reclamation 2011a)

(4) The USACE is currently in the process of developing regional climate change literature syntheses at the two-digit Hydrologic Unit Code (HUC2) scale.

(5) Other sources of peer-reviewed information that are available at regional or local scales should be explored and included if appropriate to the particular scales and variables of the hydrologic study.

b. Hydrologic simulations using the bias-corrected, spatially disaggregated (BCSD) archive and the Variable Infiltration Capacity (VIC) hydrologic model are appropriate and available through http://gdo-dcp.ucllnl.org/downscaled_cmip_projections/dcpInterface.html. These data were produced by USACE in conjunction with Lawrence Livermore National Laboratory, the Bureau of Reclamation, the U.S. Geological Survey, Climate Central, Scripps Oceanographic Institute, and Santa Clara University as described at the online archive.

c. Hydrologic information developed for the USACE screening-level watershed-scale vulnerability assessments at the HUC-4 scale.

d. If available in the region, other USACE analyses that include climate change information can also be used. For example, USACE climate change adaptation pilots may have developed regional to local information that addresses climate change hazards or vulnerabilities.

4. Evaluation of Phase II Information. A robust evaluation of available information encompasses present patterns of climate change as well as future projected climate changes expected to impact watershed hydrology in the project region.

a. The literature evaluation should include a description of each source along with:

(1) The length and quality of the observed record;

(2) Any statistically significant trends in the observed record for the hydrologic variables of interest or underlying physical processes;

(3) The type and quality of the projected climate information related to the hydrologic variables of interest or underlying physical processes;

(4) The direction and (if available) magnitude of the projected relevant changes, as well as any projected trends.

b. Similarities and differences in the literature should be noted, with a discussion about how these might be considered in project planning and design. In cases where information from the literature conflicts, these results could be considered to provide a range of potential future conditions without assigning weights or expected probabilities to those potential futures. It is important that the qualitative analyses do not inject false precision by prematurely down-selecting to a limited set of the available projected future conditions.

c. Where applicable, a first-order statistical analyses of readily available projected climate data may be performed using standard statistical methods to characterize the data and identify trends for variables relevant and at a scale appropriate to the hydrologic study.

5. Example Qualitative Analysis. The example qualitative analysis is for a Flood Risk Management project in northeastern Kansas, in HUC 1027 (Kansas: The Kansas River Basin, excluding the Republican and Smoky Hill River Basins. Kansas, Nebraska, Missouri).

a. Project Description. A system of levees currently in place is being studied for possible modifications to achieve additional project goals for flood risk reduction. The no-action alternative is to maintain the levee system as it currently exists. A study is being conducted to evaluate the feasibility of raising levee heights at certain locations to provide additional flood risk reduction. The hydrologic analysis is directed at updating estimates of flood frequency. The existing flood frequency information was last investigated for a period of record ending in the 1960s. Since that time, several floods have occurred, including the 1993 flood of record. Increases in projected future flood magnitude and frequency could impact both the future with- and without-project conditions, and may result in different benefits compared to the without-climate change analysis. Increases in future flood magnitude or frequency could also alter project performance, including increased maintenance costs or repairs associated with overtopping events that are potentially more frequent than originally assumed.

b. Phase I Qualitative Assessment. The flood reduction project is intended to reduce damage associated with flood events in northeastern Kansas in the vicinity of the Big Blue River. Any future conditions which increase the magnitude or frequency of flood flows would impact the project. Therefore, climate change is a consideration for this project.

c. Phase II Identification of Climate Threats and Impacts.

(1) Observed Record. For the period of record from 22 July 1959 through 21 January 2010, daily observations of discharge for inflow at the project site were analyzed in two ways. The first method involved performing a linear regression of the annual maximum daily discharge from the record to determine if there is a statistically significant slope (Figure C-1). Simple linear regression with test statistics can be performed using the method of least squared errors in a variety of software programs, including Microsoft Excel's "Analysis Toolpack" -

"Regression" macro. The second method involves performing a linear regression of the largest annual three-day maximum discharge to determine if there is a statistically significant slope (Figure C-2). Both analyses resulted in a relatively small but statistically significant trend at the $p<0.05$ level towards smaller annual maximum daily discharges and smaller annual maximum three-day average discharges.

(2) Projected Future. The NOAA National Environmental Satellite, Data and Information Service (NESDIS) released a report in January 2013 that assessed climate trends and scenarios into the next 50–100 years for the Great Plains region (NOAA 2013). The report indicates that over the period of hydroclimatological record for northeast Kansas, both temperature and precipitation have trended above normal, especially over the last 50 years. To account for climate change, the forecast of future meteorological conditions in the region considers the past temperature and precipitation records, as well as the modeled future conditions in the area through 2070. According to the NESDIS report, a warming trend of about 3–5°F and a precipitation trend toward slightly wetter conditions can be expected over the next 50 years, although these estimates have significant uncertainty. Numerous reputable and peer-reviewed climate change syntheses, including Kunkel et al. (2013), suggest that a warming climate can increase the risk of very heavy precipitation and flooding. The USACE screening-level watershed vulnerability assessment for HUC 1027 showed that this watershed is in the 20% most vulnerable for the flood risk reduction business line for the wet scenarios, primarily due to the cumulative flood magnification factor (FMF, Vogel et al 2011). The cumulative and local FMF computed for the watershed (as of March 2014) are greater than 1.0 for both wet and dry future conditions (i.e., flood magnitudes are expected to increase in the future).

(3) An additional analysis was performed to provide first-order detection of any changes in floods for both the observed record and the projected future based on bias-corrected and spatially downscaled data from simulations developed for the Coupled Model Intercomparison Project Phase 5 (CMIP5) data, with hydrologic response simulated by the Variable Infiltration Capacity (VIC) model (Liang et al. 1994) at http://gdo-dcp.ucllnl.org/downscaled_cmip_projections/dcpInterface.html

(i) The first-order statistical analysis for the 100 simulations for 1950 to 1999 indicates no statistically significant linear trend for potential realizations of runoff for the 20th century (Figure C-3). Note that this is simply a review of modeled conditions and does not use actual measurements for that time period. The actual measurements are shown in Figure C-1.

(ii) A statistical analysis of the projected hydrology for 2000 to 2099 indicates a statistically significant linear trend of increasing average annual maximum monthly flows (Figure C-4). This trend is consistent with the literature, which indicates that floods may increase in this area in future.

d. Conclusion of Phase II Evaluation: Although the observed trend indicates a slight decrease in runoff for the period of record at the example location, the literature consistently projects a trend toward increasing runoff. The USACE screening-level watershed vulnerability assessment indicates that the FMF is slightly greater than 1.0 even in a drier future. The first-

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order analysis of projected future conditions indicates that climate change in the next 50 years may increase flood flow frequency in the study basin. Based on the assessment, which shows differing but relatively small signals, the recommendation is to treat the potential increases in flood magnitude as occurring within the uncertainty range calculated for the current hydrologic analysis.

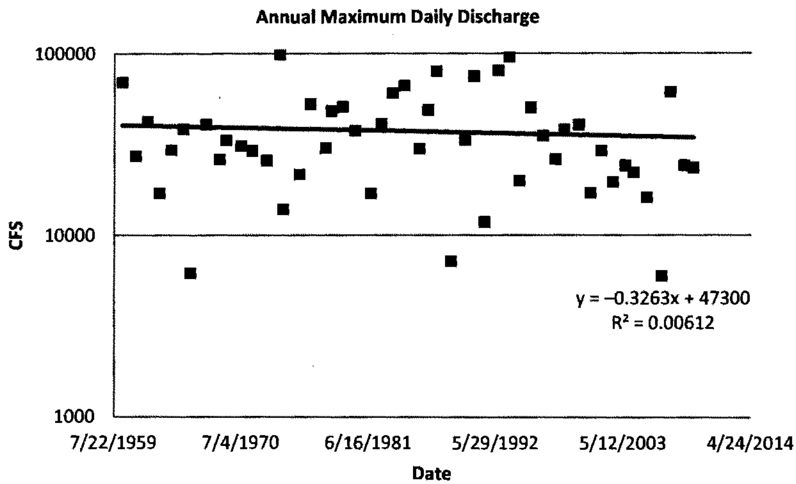

Figure C-1. First-order trend detection for observed annual maximum daily inflows in the example region of northeastern Kansas. A negative slope is determined to be statistically significant at the p<0.05 level.

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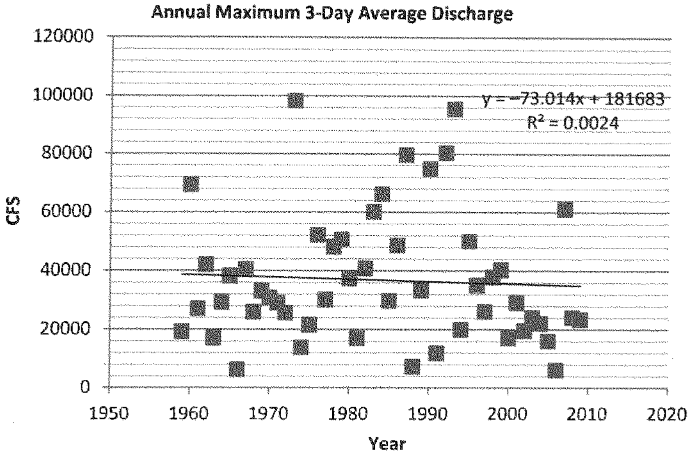

Figure C-2. First-order trend detection on observed annual three-day maximum daily inflows in the example region of northeastern Kansas. A negative slope is determined to be statistically significant at the p<0.05 level.

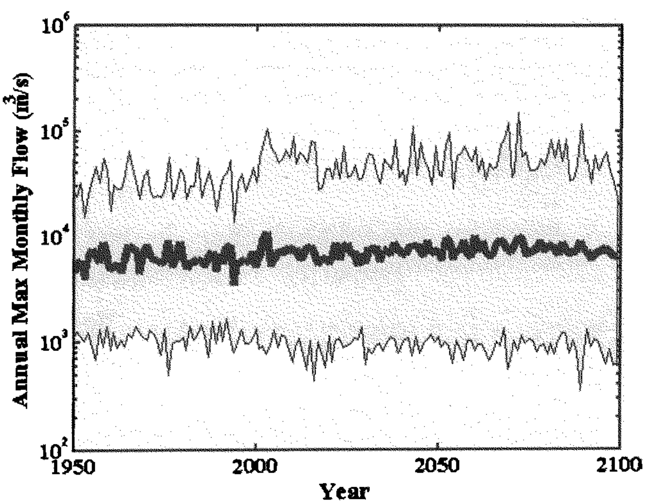

Figure C-3. Projections of climate-changed hydrology for HUC 4 1027. The mean of 100 projections of annual maximum monthly flow is in blue and the range of those 100 projections is in yellow.

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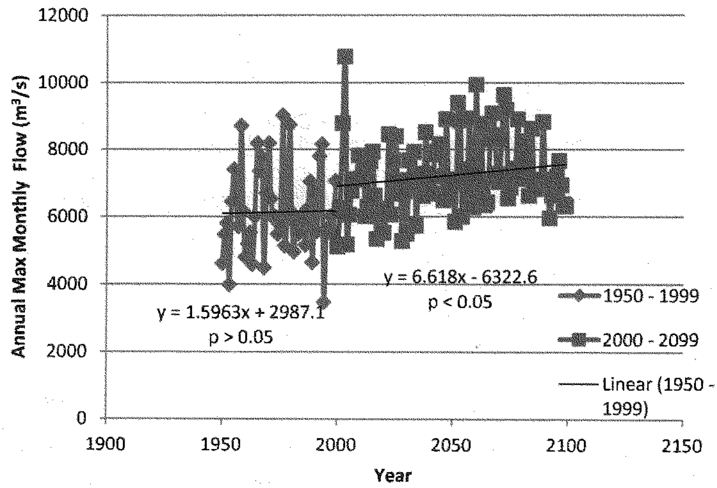

Figure C-4. Statistical analysis of the mean of the annual maximum monthly flow projections. The 1950–1999 period has no statistically significant trend, but the trend for 2000–2100 is statistically significant at the p<0.05 level.

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Appendix D: Preview of Quantitative Analysis Requirements.

1. Quantitative Climate Change Analysis for Hydrologic Analyses in Planning and Engineering Design Studies. Quantitative assessments are necessarily project-specific and will be conducted explicitly for impacts to the authorized purposes of the project. The outputs from a quantitative analysis can directly alter the numerical calculations and results in the hydrologic analysis. The amount of alteration is determined by the amount of evidence indicating that climate change is affecting the hydrologic metric of interest in the present and future. These changes to numerical results can alter calculations of project benefits and costs, thus directly informing the decision process. The quantitative assessment to be required in future will require different processes for uncertainty assessment. These will be described in future additions to this guidance along with new information for considering those climate-related uncertainties in the context of other uncertainties associated with the hydrologic estimates under future conditions.

a. Specific guidance for implementing quantitative analyses will be provided as methods are developed. This guidance will be developed based on project type (e.g., Flood Risk Management, Navigation, Water Management, Levee Safety). Once additional guidance is provided for specific project types, a quantitative analysis will be required in addition to the qualitative analysis when at least one of the following is true:

(1) The qualitative analysis indicates an expectation that consideration of climate change will alter hydrologic analyses and potentially affect the decision outcome, OR

(2) Feasibility Phase: The TSP milestone has not yet been completed, OR

(3) PED Phase: The required hydrology and hydraulics components of the PED phase are less than 50% complete, as of the date of issuance of project-type quantitative guidance ECs.

b. The three primary components of any future quantitative guidance will be detection of trends, attribution of these trends to climate change, and projection of future trends.

(1) Detection. The first step in a quantitative analysis is to attempt to detect changes in the observed hydrologic record for the metric relevant to the study, such as increases or decreases in variability or magnitude (see Kundzewicz and Robson (2000) for examples). If no change is detected, no further quantitative analysis will be necessary. USACE is developing information and inputs to forthcoming guidance which will support methods of detection to be required in the quantitative analyses at a later date. This information will be distributed together with the future guidance requirements as described above.

(2) Attribution. If a change is detected through statistical analysis, the next step is to attempt to attribute the change to one or more causes, primarily by evaluating additional information about changes in the watershed, searching the supporting literature, and in some cases using results from experiments with numerical climate simulation models already performed – no new numerical climate simulations will be required. Hegerl and Zwiers (2011)

provide a review of possible attribution strategies and discuss the difficulties in attributing changes using only observational data. As with the detection methods, for attribution, USACE is developing information to support its application in the quantitative analyses to be required in future. This information will be distributed together with the future guidance requirements as described above.

(3) Projection. Finally, projected hydrologic changes are analyzed. Climate projections such as those available at http://gdo-dcp.ucllnl.org/downscaled_cmip_projections/ can be used in concert with hydrologic simulation tools to obtain projections of specific hydrologic variables. Well-documented and peer-reviewed models have been applied to assess climate change impacts in many locations and at many scales in the US. These applications include use of HEC-HMS, the Variable Infiltration Capacity Model (VIC) (Christensen et al. 2004; Payne et al. 2004; Christensen and Lettenmaier 2007; Maurer 2007; Barnett et al. 2008; McGuire and Hamlet 2010; Bureau of Reclamation 2011b), the Sacramento Model (SAC-SMA) (Brekke et al. 2009; Raff et al. 2009; Maurer et al. 2010), and others.

US Army Corps
of Engineers
Kansas City District
Northwestern Division

Kansas Citys, Missouri and Kansas Flood Risk Management Project

Final Feasibility Report

May 2014

RECORD OF DECISION

KANSAS CITYS, MISSOURI AND KANSAS FLOOD RISK MANAGEMENT PROJECT

The Final Feasibility Report for Kansas Citys, Missouri and Kansas Flood Risk Management Project (Phase II), dated May 2014, addresses flood risk management problems for the second phase of the Interim Feasibility Report and Environmental Impact Statement (IFR/EIS) (Phase I), dated August 2006 with addendum dated December 2006. The Phase I report and EIS analyzed alternatives and environmental impacts for the seven levee units that compromise the Kansas Citys Local Flood Risk Management Project including the Argentine, East Bottoms, Fairfax-Jersey Creek, Birmingham, North Kansas City, Armourdale and Central Industrial District (CID) levee units. The Phase I report identified a recommended plan for five of the seven levee units: the Argentine, Fairfax-Jersey Creek, North Kansas City, East Bottoms and Birmingham levee units. The Phase I report resulted in a report of the Chief of Engineer's (Chief's Report), dated December 19, 2006, and a Record of Decision (ROD) dated November 21, 2007. The impacts to the environment from potential alternatives at Armourdale and CID levee units, including the ultimately recommended plan for these segments, were analyzed in the Phase I IFR/EIS. However, the Phase I ROD only addressed the decision for the Argentine, Fairfax-Jersey Creek, North Kansas City, East Bottoms and Birmingham levee units.

The Phase II feasibility report and the Phase I IFR/EIS addresses the effects of the flood risk management improvements on the natural system and the human environment for the Armourdale and CID levee units. The final recommendation is contained in the Chief's Report, dated January 27, 2015. The Phase II feasibility report, Phase I IFR/EIS, and the Chief's Report are incorporated herein by reference. Based on these reports, the reviews of other Federal, State and local agencies, input from the public, and review by my staff, I find the plan recommended by the Chief of Engineers to be technically feasible, economically justified, in accordance with environmental statutes, and in the public interest.

The recommended plan is the Locally Preferred Plan and includes:

- Improvements to the CID levee unit on the Kansas River:
  - Increasing the levee unit height of approximately 11,750 linear feet of levee and floodwall between 0.2 and 3.8 feet (average increase 3.6 feet);
  - Constructing an additional 600 linear feet of new floodwall;
  - Installing underseepage control features including 57 relief wells and approximately 3,450 linear feet of area fill;
  - Installing four new closure structures and modifying or replacing two closures to the CID levee unit,
  - Modifying five pump stations and removing two stations,
  - Modifying drainage structures, and relocating utility crossings; and,
  - Modifying approximately 290 linear feet of floodwall to improve structural reliability.

- Improvements to the Armourdale Levee Unit on the Kansas River:
  - Increasing the levee unit height of approximately 33,000 linear feet of levee and floodwall between 1.2 and 5.2 feet (average increase 4 feet),
  - Installing underseepage control measures including 74 relief wells and 2,000 linear feet of underground slurry cutoff wall;
  - Installing three closure structures and modifying or replacing four closures,
  - Modifying seven pump stations and removing two stations, and,
  - Modifying drainage structures, and relocating utility crossings.

The alternatives identified, evaluated and recommended in the Phase II feasibility report are the same scope and location as addressed in the Phase I IFR/EIS. In addition to the "no action" plan, six alternatives for the CID levee unit and various alternatives for each of the seven reaches of the Armourdale levee unit were evaluated. The differences among the CID alternatives are related to a proposed new tieback measure; whether or not a new tieback is included, where the tieback connection is located along the existing alignment, the effect of the new tieback on the proposed relief well system, and the alignment of the tieback between the existing unit and the bluff. For most reaches of the Armourdale Unit, only one alternative plan was identified as technically feasible and effective to perform the raise and address the respective impacts to appurtenant structural and geotechnical features. However, multiple alternative plans for structural modifications were identified and evaluated for five reaches within the Armourdale Unit. For these reaches, the alternatives that avoid encroachments and impacts to adjacent businesses and known hazardous, toxic, and radioactive waste areas were included in the recommended plan. The non-structural measures considered included structure removal or relocation, structure elevation, and structure flood proofing. The non-structural measures were eliminated as they were determined not to be efficient or effective at managing the flood risk.

The effects of the recommended plan do not exceed the effects discussed in the Phase I IFR/EIS. Accordingly, impacts to natural resources are minor and may include the removal of some grasses, weeds and incidental seedlings. In these Armourdale and CID levee units, impacts to mature trees and wetlands are not anticipated. All practicable means to avoid and/or minimize adverse environmental effects have been incorporated into the recommended plan. No compensatory mitigation is required. The recommended plan is the environmentally preferred alternative.

In addition to the public review process conducted for the Phase I IFR/EIS, the public review was conducted on the Phase II feasibility report. The public review of the draft Phase II feasibility report was completed on December 21, 2013. All comments from the public are addressed in the final Phase II feasibility report. The state and agency review of the final Phase II feasibility report and draft Chief's Report was completed June 22, 2014. Comments from state and Federal agencies did not result in any changes to the final Phase II feasibility report.

The combination of recommendations from the Phase II feasibility report and the Phase I IFR/EIS represent a complete and complimentary effort that addresses the existing Kansas Citys Flood Risk Management System as a whole. The two phases of the study effort have maintained a consistent approach to improving performance and reliability within the system. Although the Phase I IFR/EIS recommendations have previously been authorized, it is important to recognize the overarching systems approach to metropolitan flood risk management by evaluating the two sets of recommendations together and presenting a total system recommendation. When Congress authorized Phase I of the project pursuant to section 1001 of the Water Resources Development Act of 2007 (Public Law 110-114), it was understood that Phase I was a partial response to addressing the flood risk management problems in the system. The Phase II recommended plan is necessary to more completely provide for system wide flood risk management benefits.

Technical, environmental, economic and cost-effective criteria used in the formulation of alternative plans were those specified in the Water Resource Council's 1983 Economic and Environmental Principles and Guidelines for Water and Related Land Resource Implementation Studies. All applicable laws, executive orders, regulations, and guidelines were considered in evaluating alternatives. Based on review of these evaluations, I find that the recommended plan benefits outweigh the costs and any adverse effects. This Record of Decision completes the National Environmental Policy Act process.

March 28, 2016
Date

Jo-Ellen Darcy
Jo-Ellen Darcy
Assistant Secretary of the Army
(Civil Works)

Kansas Citys, Kansas and Missouri Flood Risk Management Project

Final Feasibility Report
May 2014

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EXECUTIVE SUMMARY

The existing Kansas Citys, Missouri and Kansas, Flood Risk Management Project provides local flood risk management for the metropolitan areas of Kansas City, Missouri, and Kansas City, Kansas. The Kansas Citys project is authorized as a system of seven levee units. This project extends over the lowest 10 miles of the Kansas River (at its confluence with the Missouri River) and a 20 mile reach of the Missouri River flanking the mouth of the Kansas River. The Kansas Citys project is a unit of the Missouri River basin comprehensive plan authorized and modified by the 1936, 1944, 1946, and 1954 Flood Control Acts. The last major modification to raise some of the levee units comprising the Kansas Citys Project was authorized in 1962.

Section 216 of the 1970 Flood Control Act provides the authority to reexamine a completed civil works project and recommend modifications or improvements. An Interim Feasibility Report and an Environmental Impact Statement (EIS), published in September 2006, recommended performance improvements in four of the units: Argentine, North Kansas City, Fairfax-Jersey Creek, and East Bottoms. These recommendations were subsequently authorized by the Water Resources Development Act of 2007 and have proceeded with design and implementation. The Interim Report concluded that no improvements were needed in the Birmingham Unit.

This Final Feasibility Report addresses the remaining two levee units; the Armourdale and Central Industrial District (CID) Units. The EIS published with the Interim Report included analyses for all seven levee units, including the two units addressed in this report. The alternatives identified in this Final Report are the same scope and location as addressed in the existing EIS. A review of the current environmental conditions in the study area confirmed that no significant changes have occurred since 2006. No new NEPA documentation is required.

The Armourdale Unit is located in Wyandotte County Kansas, along the left bank of the Kansas River from mile 7 (Mattoon Creek) to mile 0.3, near the confluence of the Kansas and Missouri Rivers. The primary components of the unit consist of earthen levees, floodwalls, riprap and toe protection on riverward slopes of levees, toe drains along the concrete floodwalls, sandbag gaps, stoplog gaps, drainage structures, relief wells and pumping plants. The floodwalls, in two reaches, vary from 11 to 17 feet high and total approximately 6,200 feet. The levees, in three reaches, vary from 4 to 17 feet high and total about 5.3 miles. Existing underseepage control features include approximately 13,400 LF of riverside impervious fill cutoffs, 1,550 LF of landward underseepage berm, and 39 relief wells with collector systems in several reaches.

Although the CID Unit is one continuous levee unit, it crosses the Kansas and Missouri State Line and is subsequently operated and managed as two separate and distinct sections: the CID-Kansas section, and the CID-Missouri section. The CID-Kansas Section (CID-KS), is located in Wyandotte County, Kansas, and extends along the right bank of the Kansas River from mile 3.4 to the mouth, then downstream along the right bank of the Missouri River to the State Line. The unit consists of two levee reaches, three floodwall reaches, riprap and levee toe protection, a surfaced levee crown and ramps, a stoplog gap, a sandbag gap, eight pumping stations, drainage structures, and relief wells. The levees total approximately 1.7 miles long and the floodwalls about 7,900 feet. The section varies from zero to 14.5 feet high. Existing underseepage control features in CID-KS includes a buried collector system, approximately 1,800 LF of area fill, and 19 relief wells with collector system.

The CID-Missouri section (CID-MO) is located in Jackson County, Missouri. This section extends along the right bank of the Missouri River from near the Kansas-Missouri state line, river mile 367.2, and ending near river mile 365.7. The CID-MO section consists of levee, floodwalls, a levee drainage system and pumping plants, sandbag and stoplog gaps, toe and bank protection, and slope protection on the riverward slope. The CID-MO section floodwalls total 1.5 miles and the levee is about 430 feet.

The feasibility study assessments provide insight into both the existing levee performance and the economic damages expected under existing conditions for an array of high water events. Much of the analysis used data and observations from recent high water events, especially those in 1993 and, to a lesser degree, 2011. Risk and uncertainty analysis results and observations of levee performance during flood events form the basis for the identification of opportunities for risk reduction measures. The critical reaches for geotechnical underseepage failure and slope stability risks were identified and analyzed in each unit. Probabilities of failure versus water surface elevation were calculated for the most critical features to determine the overall existing risk for each unit. The current existing failure risk, in terms of annual exceedance probability, is significantly high in both units: 3.5% for the Armourdale Unit and a 0.3% for the CID Unit.

Deficiencies in multiple pump stations are the major contributors to the existing condition probability geotechnical/structural failure, which would cause a breach in the Armourdale Unit. For the CID Unit, it is structural gatewells, floodwalls, and stoplog gaps that contribute to a lesser, but still significant, probability of structural/geotechnical failure, which would cause a breach. Both the Armourdale and CID units have some probability of breach under existing conditions, but the probability of breach is much greater for the Armourdale Unit. For both units, the analysis indicates that the unit will structurally fail prior to overtopping.

If all geotechnical and structural failure risks were addressed, a significant overtopping risk would still remain for the target 0.2% chance flood event. These findings for overtopping risk in the lower Kansas River show that these units do not reliably achieve the authorized 390,000 cfs conveyance target. This indicates the need for a general increase in the existing overtopping protection along the lower Kansas River.

Management measures considered to address the identified conditions of unacceptable flood risk include: no action, non-structural, and structural measures. The selection of management measures and development of alternatives focuses on achieving and maintaining a uniform level of flood risk management for the Kansas Citys system. The maximum target system performance level has been selected as the elevation three feet above the 0.2% flood event water surface profile. The management measures for structural and geotechnical components were evaluated for their feasibility and effectiveness under the hydraulic conditions expected at the desired top of levee elevation. Alternative plans were evaluated, which included modifications to floodwalls, levees, underseepage controls, pump stations, and unit tiebacks.

Six alternative plans for the CID Unit were retained for the final evaluation. Each plan includes the same raises of the earthen levee and floodwall sections, the same area fill locations, and the same pump station modification and abandonments. The differences among the plans are related to a new tieback measure; whether or not a new tieback is included, where the tieback connection is located along the existing alignment, the effect of the new tieback on the proposed relief well system, and what alignment the tieback is constructed on between the existing unit and the bluff.

For most reaches of the Armourdale Unit, only one alternative plan was identified as technically feasible and effective to perform the raise and address the respective impacts to appurtenant structural and geotechnical features. Additionally, multiple alternative plans for structural modifications were identified and evaluated for five reaches within the Armourdale Unit.

For the purpose of plan selection, economic analysis was conducted to develop a risk-based evaluation in terms of benefits, costs, and performance of the alternatives under the future with-project condition. The analysis encompasses all flood-prone properties within the study area. All costs include interest during construction computations which assume project completion in mid-2026. All costs reflect an October 2013 price level and the annualized totals reflect the current Federal interest rate of 3.5 percent as well as a 50-year period of analysis. OMRR&R costs were included in this analysis for those features that will incur a net cost over and above present levels.

The Final Feasibility Report Recommended Plan generates annual flood risk management benefits of $56.7 million at an annual cost of $16.8 million. Net average annual equivalent benefits are $39.9 million and the benefit to cost ratio is 3.4 to 1. Each unit is also individually justified.

With net benefits of $39.9 million, the recommended plan represents a strong contribution to national economic outputs and is shown to be an efficient project meeting the planning objectives, constraints, and criteria; limiting land disturbance and environmental impacts; and avoiding HTRW disturbances and significant real estate conflicts and relocations. Considering the urban industrial nature of both areas, it is possible that unidentified concerns are present. Additional soil sampling and testing will be conducted as part of the design phase, as well as close monitoring of material excavated during the project construction, to ensure that any HTRW uncovered is properly handled and disposed. Any and all removal of contaminated soils or other contaminated materials found will be 100% local sponsor responsibility (including cost). All removal of contaminated soils or other contaminated materials must be completed prior to construction.

The median annual exceedance probability – currently as much as 3.5 percent for Armourdale and 0.3 percent for CID in their existing conditions – would improve to 0.12 percent for both units. In other words, there would be a 0.12 percent chance of a damaging flood in any year following project implementation. Under existing conditions, in the 1 percent annual chance flood event, both units would have between an 11 percent and 55 percent annual chance of experiencing damage due to overtopping or breach failure. These probabilities are improved to roughly 1 percent in the future-with-project condition.

The long-term with-project risk of a damaging flood in both of the units over 50-year period would be less than 1 in 10, compared to a current 50-year risk exceeding 1 in 2 in Armourdale and approximately 1 in 5 in CID.

Under the Final Feasibility Report Recommended Plan, both levee units will comply with and exceed FEMA base flood insurance certification requirements (sufficient to pass the 1% event with 90% assurance). Furthermore, both units will have approximately 65-70% assurance against the median 0.2% chance exceedance flood profile

The estimated implementation cost of the Final Feasibility Report Recommended Plan is $203,711,000 Federal and $109,691,000 Non-Federal for a total estimated cost of $313,402,000 at October 2013 price levels. Project costs will be shared with two non-federal sponsors: the Kaw Valley Drainage District of Wyandotte County, Kansas (KVDD) and the City of Kansas City, Missouri. A set of maps showing the locations of the proposed modification is provided following the main report text.

The combination of recommendations from this Final Feasibility Report and the Interim Feasibility Report, approved in 2006 and authorized in 2007, represent a complete and complementary efforts that together addresses the existing Kansas Citys Flood Risk Management System as a whole. The two phases of the study effort have maintained a consistent approach to improving performance and reliability within the system.

All items included in the System Recommended Plan are necessary to continue providing flood risk management benefits as intended by Congress.

REVIEW OF COMPLETED PROJECT
KANSAS CITYS LEVEES, MISSOURI AND KANSAS
FINAL FEASIBILITY REPORT

1 Study Information

The existing Kansas Citys, Missouri and Kansas, Flood Risk Management Project provides local flood risk management for the metropolitan areas of Kansas City, Missouri, and Kansas City, Kansas. The Kansas Citys project is a unit of the Missouri River basin comprehensive plan authorized and modified by the 1936, 1944, 1946, and 1954 Flood Control Acts. The last major modification to raise some of the levee units comprising the Kansas Citys Project was authorized in 1962.

The Kansas Citys project is authorized as seven levee units. A simplified map of the system is shown in Figure 1. A more detailed system map is provided in Exhibit 1 following the report text. This project extends over the lowest 10 miles of the Kansas River (at its confluence with the Missouri River) and a 20 mile reach of the Missouri River flanking the mouth of the Kansas River. These units act in concert to manage flood risks for an area of dense industrial and commercial development and minor areas of farmland all together covering about 32 square miles. Five of the seven units protect residential development. Communities within the study area include Kansas City, Missouri; North Kansas City, Missouri; Randolph, Missouri; Birmingham, Missouri; and Kansas City, Kansas.

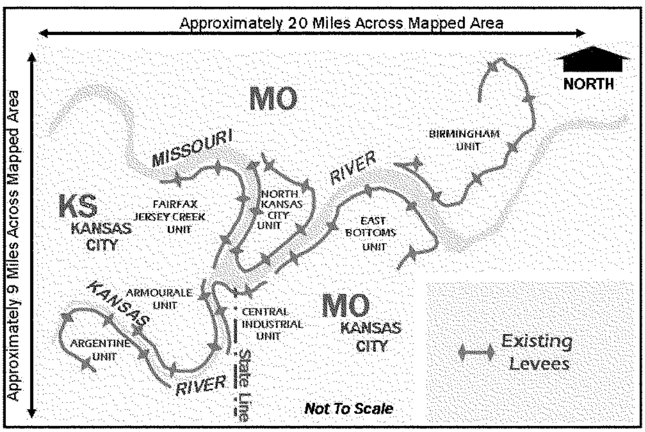


Figure 1: Simplified Kansas Citys System Map

Although the project is designed and functions as a coordinated system, its components are located on opposite banks of two major rivers within two states and various political jurisdictions. Thus, the seven levee units are operated and maintained independently by five non-federal sponsors. Most of the Federally constructed works date to the 1940's and 1950's.

Significant Federal modifications to several units were accomplished in the 1970's. While this metropolitan flood risk management system is designated as a Federal project, it has long been turned over to local sponsors for operation and maintenance. The Corps of Engineers continues to conduct regular inspections and technical review of significant modifications to the system. The entire metropolitan system of seven levee units withstood the Missouri River Flood of 1993, but some components were nearly overtopped or experienced underseepage issues. As a result, there was a concern that the levees may provide less than the intended design level of flood risk management. Section 216 of the 1970 Flood Control Act provides the authority to reexamine a completed civil works project and recommend modifications or improvements.

An Interim Feasibility Report, published in September 2006, recommended performance improvements in four of the units: Argentine, North Kansas City, Fairfax-Jersey Creek, and East Bottoms. The Interim Report concluded that no improvements were needed in the Birmingham Unit. This Final Feasibility Report addresses the remaining two levee units; the Armourdale and Central Industrial District (CID) Units.

This report focuses on identifying, describing, evaluating, and recommending alternatives to improve identified performance weaknesses in the Armourdale and CID Units by reducing the risk of flooding due to overtopping, underseepage, or structural failure.

This report also provides an update of the Interim Report recommendations and presents both sets of recommendations as a coordinated plan of system wide performance improvement.

1.1 Problem Description

Accordingly, this feasibility study identified the following problems within the study area:

- The existing system provides less than the level of performance for which it was authorized.
- Project failure due to overtopping, underseepage, or structural inadequacy, presents a significant life safety concern and will cause catastrophic damage to the urban development in the study area.
- The existing system includes components between forty and seventy years of age. While the system has been well maintained and is currently in good working condition, the state of the art of design, construction, and reliability analysis has changed significantly since the original construction. This concern will continue to grow as the system ages.

1.2 Study Authority

Section 216 of the 1970 Flood Control Act provides continuing authority to reexamine completed civil works and determine whether the projects are providing benefits as intended. Section 216 reads as follows:

The Secretary of the Army, acting through the Chief of Engineers, is authorized to review the operation of projects, the construction of which has been completed and which were constructed by the Corps of Engineers in the interest of navigation, flood control, water supply, and related *purposes, when found advisable due to the significantly changed physical or economic conditions, and to report thereon to Congress with recommendations on the advisability of modifying structures or their operation, and for improving the quality of the environment in the overall public interest.*

The Feasibility Study began in September 2000 with the execution of a Feasibility Cost Sharing Agreement between the Corps of Engineers and the local non-Federal levee sponsors. The study is cost-shared 50% Federal and 50% non-Federal.

1.3 Purpose and Scope

The purpose of the feasibility study effort is to review the conditions of the existing flood risk management system, identify potential weaknesses and areas of concerns, and analyze alternatives for potential improvements to increase the project performance and reduce the risk of flooding to local communities. In order to enable the study of the overall system to progress in an efficient and orderly manner within available funding, the study was separated into Phase 1 and Phase 2 efforts in 2006. This two-step reporting process meant the complete feasibility study would generate two sets of recommendations.

At the time the phasing decision was made, hydrology and hydraulics modeling and analysis was complete for the entire system. However, structural and geotechnical analysis and calculations were not complete for all units. Those units wherein the analyses were complete were included in Phase 1 (the Argentine, North Kansas City, East Bottoms, Fairfax-Jersey Creek, and Birmingham Units). Those units for which the level of detail desired was not yet fully developed, or significant uncertainties remained, were included in Phase 2 of the study for further evaluation (the Armourdale and Central Industrial District (CID) Units).

Phase 1 and Phase 2 are complementary efforts that view the Kansas Citys project as one complete system. This Final Report documents the existing conditions, evaluation of alternatives, and improvement recommendations for the two units addressed in Phase 2 (Armourdale and CID). Additional details on the other units of the system and their recommendations are provided in the Interim Report. Historical and reference information on the entire system, and updates to information presented in previous report if applicable, are provided in this Final Report where needed for context and continuity. Additionally, this Report presents the combination of Phase 1 and Phase 2 of the study as an integrated economically justified plan for improvement to the overall system.

1.4 Study Area and Non-Federal Sponsors

The overall feasibility study effort addresses the areas within the existing seven units of the Kansas Citys system and directly affected adjacent areas. Within this Final Feasibility Report, the terms "study area" and "project area" refer only to the Armourdale and CID units, unless specifically noted otherwise.

The Phase 2 study has been conducted in conjunction with two sponsors: the Kaw Valley Drainage District of Wyandotte County, Kansas, and the City of Kansas City, Missouri. The

Kaw Valley Drainage District (KVDD) is the owner-operator of the Armourdale Unit and the Kansas Section of the CID Unit. The City of Kansas City, MO (KCMO) is the owner-operator of the Missouri Section of the CID Unit.

1.5 History of the Investigation

The entire metropolitan system withstood the Missouri River Flood of 1993, but the general performance of the system was severely tested as the flood crest reached within one foot of overtopping in at least one location. Not only were stages extreme, but durations were lengthy. Concerns arose about the reliability of the system to prevent overtopping and adequately handle underseepage. Further, there was a concern that the levees may provide less than the original authorized and intended level of performance.

The Kansas Citys metropolitan population and economy have grown significantly since the last system improvements were authorized in 1962. Much of the metropolitan economy is dependent on the areas within the levee system. Parts of the existing system are well over 60 years old. Project failure would endanger lives and create massive physical flood damages.

Both natural and man-induced geomorphologic changes have occurred since the last project authorization. Reservoirs have reduced some of the river systems' sediment load and navigation structures as well as natural river processes have contributed to the Missouri River's cross-sectional adjustments.

In response to the performance observed in 1993, both Kansas City, Kansas, and Kansas City, Missouri, wrote letters to the Kansas City District expressing concern for the adequacy of the system. A Reconnaissance Report was prepared and published in August 1999 which found that there was a Federal interest in proceeding with a Feasibility Study. The Reconnaissance Report was approved by Corps of Engineers Headquarters in July 2000.

The Phase 1 study effort resulted in the Interim Feasibility Report (Interim Report) and an Environmental Impact Statement (EIS), published in Aug 2006. These recommendations were subsequently authorized by the Water Resources Development Act of 2007 and have proceeded with design and implementation.

The EIS published with the Interim Report included analyses of the existing environmental conditions and potential impacts of project implementation in all seven levee units. This Final report covers two of the seven units addressed in the EIS. The alternatives identified and recommended in this Final Report are within the same footprint of disturbance for the tentatively recommended plan identified in the EIS. The plans recommended in this Final Report contain additional design refinements, which fall within the resources, location, and impacts addressed in the EIS.

1.6 Existing Projects and Prior Reports

The existing Kansas Citys project was created and subsequently modified by the Flood Control Acts authorized in 1936, 1944, 1946, 1954, and 1962. Following the 1936 Flood Control Act, construction of the first Federal levees began around 1940. The original Federal construction included some incorporation of, and improvements to, previously existing local levees. Much of the authorized system was nearing completion at the time of the 1951 Kansas River Flood. In this catastrophic flood, the Argentine, Armourdale, CID, and Fairfax levees were overtopped and heavily damaged. Based on this experience, Congress later authorized the Kansas River basin reservoir system in the 1954 Flood Control Act.

The Kansas Citys system, especially along the Kansas River, was re-examined during the post-1951 period as the Kansas River basin reservoirs were being designed and constructed. This led to a major modification (raise) of the Armourdale, Argentine, and CID Units authorized by Public Law 87-874 on October 23, 1962 (the "1962 modification"). Construction of these modifications began in 1971.

The modified design of the Kansas Citys project, including the authorized design discharges for the Kansas River levee units, was predicated on construction and operation of the Kansas River Basin system of reservoirs as authorized in 1954. Most of the lakes in that system are in place and operating, but three of the smaller originally authorized lakes in the system (Woodbine, Grove and Onaga) were not built.

The existing protective works consist principally of levees, floodwalls, bridge and approach alterations, and some limited channel improvement and alteration. The project extends over the lower 10 miles of the Kansas River and on the Missouri River from 6.5 miles upstream to 12.5 miles downstream of the mouth of the Kansas River. The 32-square-mile study area includes the heavily industrialized floodplains of the two rivers. Complete effectiveness of the overall project is contingent upon adequate reservoir control in the upper Missouri and Kansas River basins.

The existing levee system and its components were authorized by specific legislation, as documented in multiple reports of Congress, and have been implemented through a series of definite project reports (DPR's), design memorandums (DM's), and operations and maintenance (O&M) manuals. Following original project implementation, multiple reports and studies have been prepared and published at various times including reservoir regulations, post-flood assessments, river hydrology updates, and flood plain hazard evaluations. The Interim Feasibility Report and the associated EIS (both August 2006), reviewed and incorporated information from the multitude of prior reports and are both directly referenced in this Final Feasibility Report.

<u>Review of Completed Project, Kansas Citys Levees, Missouri and Kansas, Interim Feasibility Report, USACE Kansas City District, August 2006</u>
The Interim Feasibility Report recommended improvements to four units of the system: the Fairfax-Jersey Creek, North Kansas City, and East Bottoms Units on the Missouri River, and the Argentine Unit on the Kansas River. The Missouri River Units were determined to have adequate height to resist overtopping at the design flood level, but require significant underseepage and structural modifications to maintain acceptable overall system reliability. In addition to similar geotechnical and structural reliability concerns, the entire Kansas River portion of the system was determined to be of insufficient height to provide adequate overtopping protection. The Interim Report included the detailed analysis of alternatives for the

Argentine Unit. The Argentine NED plan was identified as a unit raise to provide improved reliability to pass the 0.2% annual chance (500-year) plus 3 feet water surface profile. This level of flood risk management benefit is consistent with the Missouri River units, and meets economic project justification criteria.

The Armourdale and CID Units are located immediately downstream of the Argentine Unit. In order to achieve the desired condition of a uniform system level of flood risk management benefit, and to reduce the potential for induced damages between units within the system, it was determined that the development of alternatives for these two units would not consider measures providing a level of risk management or reliability greater than the authorized plan for the upstream unit.

<u>Final Environmental Impact Statement, Kansas Citys, Missouri and Kansas Flood Damage Reduction Study, Missouri and Kansas Rivers, USACE Kansas City District and U. S. Environmental Protection Agency Region VII, August 2006</u>
The seven levee units addressed in the Final Environmental Impact Statement (FEIS) include North Kansas City; Northeast Industrial District (East Bottoms) and Birmingham units in Missouri and the Argentine, Armourdale, and Fairfax-Jersey creek units in Kansas. The Central Industrial District (CID) levee unit, which protects land in both Kansas and Missouri, was also addressed in the FEIS.

Engineering, economic, and environmental analyses were conducted for the North Kansas City, East Bottoms, Argentine, and Fairfax-Jersey City levee units. The results of these analyses and recommendations to increase levee unit reliability are included in the Interim Feasibility Report and the FEIS. Analysis of the Birmingham unit found no geotechnical or structural deficiencies. Therefore no reliability improvements were proposed for this unit.

Preliminary engineering, economic, and environmental analyses were conducted for the CID and Armourdale units for the FEIS. Findings for overtopping risk and geotechnical/structural risk indicated the need to pursue reliability improvements for the Armourdale and CID levee units. Projected improvements, which were assessed in the FEIS, included earthen levee raises, floodwall raises, and underseepage improvements. Tentative preferred alternatives for these units were recommended, and their environmental effects assessed, within the FEIS. The final preferred alternatives identified in this Final Feasibility Report were developed from the tentative preferred alternatives assessed in the FEIS.

1.7 Planning Process and Report Organization

The Corps of Engineers uses a six step planning process to guide project studies, as detailed in Engineering Regulation (ER) 1105-2-100 "Planning Guidance Notebook". This process is a structured approach to problem solving which provides a rational framework for sound decision making. The six steps are:

1. Identifying problems and opportunities

2. Inventorying and forecasting conditions
3. Formulating alternative plans
4. Evaluating alternative plans
5. Comparing alternative plans
6. Selecting a plan

It should be stressed that the six step process is iterative. As more information was developed throughout the study it was necessary to review and update previous steps to reach the final conclusions and analyses presented herein.

This report is generally organized to follow these six steps. The results of Steps 1 and 2 are discussed in Sections 2 and 3, respectively. Steps 4, 5, and 6 are closely related and their discussion is combined into Section 4 of the report.

2 Problem Description and Planning Objectives

This chapter presents the results of the first step of the planning process, the identification of water and related land resources problems and opportunities in the study area. The chapter concludes with the establishment of planning objectives and planning constraints, which is the basis for the formulation of alternative plans.

2.1 National Objectives and the Federal Interest

The national, or Federal, objective of water and related land resources planning is to contribute to national economic development. In addition, it must be consistent with protecting the nation's environment, pursuant to national environmental statutes, with applicable executive orders and with other Federal planning requirements. Contributions to National Economic Development (NED) are increases in the net value of the national output of goods and services, expressed in monetary units. Contributions to NED are the direct net benefits that accrue in the planning area and in the rest of the nation.

The Federal Government investigates prospective projects from a national point of view. When determining the need for Federal investment in a project, the primary analysis centers on significance of the problem and the benefits of possible solutions. In the case of this study, the focus is primarily on flood risk management benefits. It is also in the Federal and non-Federal sponsor's interest to select a cost-efficient plan, specifically one in which the benefits exceed costs. It is important to note that benefits can include non-monetary benefits such as reducing life-safety issues and improving the environmental quality. Federal interest in the project is identified when both requirements are satisfied.

Based on historical records, the study area has a high risk of flooding capable of producing significant damages and loss of life. It is within USACE and Federal interest to study the flood risk management issues within the Missouri and Kansas River Basins because there are significant risks of residential, commercial, and industrial property loss. Impacts from frequent flooding in the past include significant economic costs. Developing a project that will reduce the frequency of these damages and protect human life is within the Federal interest and a primary mission of USACE.

2.2 Public Concerns

In addition to the review of the existing project for technical or performance concerns, public input is very valuable to help define problems and opportunities. Discussion of public input and review process used in this study is included in Section 7 and Appendix G of this report.

Previous comments received with responses are documented in the Interim Report and FEIS. Comments pertinent to Phase 2 of the study were incorporated into the analysis and documented in this report. Additional comments received from the public review of this report are documented in Appendix G. The recommendation, or incorporation, of recreation features into the existing levee system is not within the Federal study authority and is left to the discretion of the non-Federal levee sponsors, so long as any such features do not hinder or conflict with the flood risk management benefits provided by the existing project. Some limited recreational features have been implemented within the Kansas Citys system.

2.3 Problems and Opportunities

Step 1 of the Planning Process seeks not only to identify the problems and opportunities within the study area, but also to establish planning objectives and constraints that will guide efforts to solve the problems and achieve the desired opportunities.

Past flood experience raised concerns that the existing system may provide less than the level of performance for which it was designed and constructed. Following the Flood of 1993, several local sponsors wrote letters to the Kansas City District expressing concern for the adequacy of parts of the flood risk management system and requesting Corps of Engineers assistance in conducting a study of the system.

In response to these local concerns, a Reconnaissance Study was undertaken through Section 216 authority. The reconnaissance study examined readily available information, data, and flood performance results, and produced recommendations supportive of a Federal Interest in proceeding with further feasibility examination. Accordingly, this feasibility study identified the following problems within the study area:

- The existing system provides less than the level of performance for which it was designed.
- Project failure due to overtopping, underseepage, or structural inadequacy, presents a significant life safety concern and will cause catastrophic damage to the urban development in the study area.
- The existing system includes components between forty and seventy years of age. While the system has been well maintained and is currently in good working condition, the state of the art of design, construction, and reliability analysis has changed significantly since the original construction. This concern will continue to grow as the system ages.

Following the problem definition, the following opportunities were identified in the study area:

- Verify current performance of the existing system versus the original design intent and project authorizations.
- Apply current understanding of large river dynamics and design criteria to assess the reliability of the existing system.
- Identify and present recommendations for designing and implementing viable measures to reduce the risk of flooding and improve the overall safety and performance of the system

2.4 Planning Objectives

Planning objectives are statements that describe the desired results of the planning process by solving the problems and taking advantage of the opportunities identified. The planning objectives will be used for the formulation and evaluation of alternative plans. They should be clearly defined and provide information on the effect desired, the subject of the objective (what will be changed, the location where the expected result will occur, the timing of the effect, and the duration of the effect. Planning objectives are listed below as they relate to each of the identified project opportunities.

- Verify current performance of the existing system.
  - Gather all available data and historical observations to develop updated engineering analysis, and combine with the economic existing conditions to establish a complete approach to estimating the existing risks and uncertainties of flood performance, reliability, and potential consequences of failure. Comparing this analysis to the authorized design and intent of the existing system will increase the knowledge and understanding of current system reliability and performance and allow the identification of areas of concern needing to be addressed by alternative measures.

- Identify and present recommendations for reducing the risk of flooding
  - Identify measures to address the identified reliability and performance inadequacies in the existing system, including hydrologic, geotechnical, and structural concerns.
  - Develop and evaluate alternatives and recommend a plan to increase the overall reliability of the existing system and reduce flood future risk and damages over the 50 year period of analysis.

- Consider the study area as a whole and thereby provide a uniform level of risk management across the system, as directed by guidance. The authorized Phase 1 project for the Argentine Unit established the formulation goal for the Kansas River units.

2.5 Planning Constraints

The Feasibility Study examination of measures to increase the performance and reliability of the system are guided by an overarching principle that seeks to achieve a relatively consistent level of performance throughout the seven unit metropolitan system. This essentially means that the study avoids recommending:

- Any measures or plans which would directly or indirectly exacerbate any performance weaknesses (or relative weaknesses) within the system, including any measure or plan that would allow one or more units of the system to provide higher or lower risk management or reliability than the rest of the overall system, and
- Any measures or plans that would contribute to increasing the level of performance of strong components of the system without a commensurate strengthening of weaker components.

Floodway Conveyance Considerations: Very early in the plan formulation process, a general guiding rule was adopted: any measures which negatively impacted the established floodway conveyance should be avoided. This was deemed essential as in most cases levees lie along both banks of the river reaches within the study area, and are often located either upstream or downstream of another unit. This principle is consistent with floodway "no rise" criteria as promulgated under FEMA regulations. This criterion was maintained during feasibility and the final alternatives are essentially benign in respect to any adverse floodway impact.

3 Existing and Future Conditions

This chapter presents the results of the second step of the planning process, inventorying and forecasting conditions. Inventorying of existing conditions is more than just describing the features of the existing project; it requires a review of the original authorization and design intent, the past performance of the system during flood events, and an assessment of the integrity of the existing system relative to current design standards. The forecasting of conditions establishes the Future Without Project scenario, to which all formulated alternative plans will be compared when assessing expected plan performance. It is important to note that the Future Without Project scenario is not the same as the existing condition, as the performance of the existing system would be expected to decline in the future if no action were taken.

3.1 Existing Unit Descriptions

3.1.1 Armourdale Levee Unit

The Armourdale Unit is located in Wyandotte County, Kansas, along the left bank of the Kansas River from mile 7 (Mattoon Creek) to mile 0.3, near the confluence of the Kansas and Missouri Rivers. Prior to the Federal project, levees and floodwalls were constructed by the Kaw Valley Drainage District. These original works were modified and expanded in the initial Federal projects. Construction of the Federal project began in 1949 and was completed in 1951. More recent improvements, separately authorized under the 1962 Modification, were completed in 1976. The levees and floodwalls of the Armourdale Unit are currently authorized to pass a maximum Kansas River flow of 390,000 cubic feet per second (cfs) coincident with a Missouri River flow of up to 220,000 cfs.

The primary components of the unit consist of earthen levees, floodwalls, riprap and toe protection on riverward slopes of levees, toe drains along the concrete floodwalls, sandbag and stoplog closures, drainage structures, relief wells and pumping plants. The floodwalls, in two reaches, vary from 11 to 17 feet high and total approximately 6,200 feet. The levees, in three reaches, vary from 4 to 17 feet high and total about 5.3 miles.

Existing underseepage control features include approximately 13,400 linear feet LF of riverside impervious fill cutoffs, 1,550 LF of landward underseepage berm, and 39 relief wells with collector systems in several reaches. Additional detail of these features is provided in Appendix A, Chapter 4. The unit begins with a stoplog closure across the Union Pacific (UP) Railroad which creates a tieback from high ground west of Mattoon Creek. The first levee section continues downstream approximately 1.28 miles along the left bank of the Kansas River, incorporating a portion of the UP embankment near the mouth of Mattoon Creek, and ends just north of the West Kansas Avenue Bridge. The first section of floodwall then extends downstream approximately 1,740 feet, ending just south of the Osage Pump Station. The second section of levee continues downstream approximately 3.3 miles to a point downstream (north) of the Chicago, Rock Island and Pacific (CRI&P) railroad bridge. This section contains one stoplog closure at the Kansas City Terminal (KCT) railroad bridge, five pumping stations, and a short reach of floodwall at the East Kansas Avenue Bridge. The second major reach of floodwall continues downstream another 4,493 feet to connect with the final levee section downstream of the Central Avenue Bridge. This section contains two sandbag closures at the UP and Missouri Pacific (MO Pac) railroad bridges, and two pumping stations. The final levee section extends another 4,156 feet and ties back into high ground at the embankment of the Lewis and Clark Viaduct.

3.1.2 Central Industrial District (CID) Levee Unit

Although the CID Unit is one continuous levee unit, it crosses the Kansas and Missouri State Line and is subsequently operated and managed as two separate and distinct sections: the CID-Kansas section, and the CID-Missouri section.

The CID-Kansas Section (CID-KS), is located in Wyandotte County, Kansas, and extends along the right bank of the Kansas River from mile 3.4 to the mouth, then downstream along the right bank of the Missouri River to the State Line. This section was originally developed by the Kaw Valley Drainage District, and initial Federal improvements began construction in 1948. Most of the Federal improvements, including repairs to damages from the 1951 Flood, were completed by 1955. The most recent improvements authorized under the 1962 Modification were completed in 1979. The CID-KS section is authorized to pass a Kansas River discharge of 390,000 cfs coincident with a Missouri River flow of 220,000 cfs.

The unit consists of two levee reaches, three floodwall reaches, riprap and levee toe protection, a surfaced levee crown and ramps, stoplog and sandbag closures, eight pumping stations, drainage structures, and relief wells. The levees total approximately 1.7 miles in length and the floodwalls about 7,900 feet. The section varies from zero to 14.5 feet high.

Existing underseepage control features in CID-KS includes a buried collector system, approximately 1,800 linear feet of area fill, and 19 relief wells with collector system. Additional details of these features are provided in Appendix A, Chapter 4.

The CID-Missouri section (CID-MO) is located in Kansas City, Jackson County, Missouri. This section extends along the right bank of the Missouri River (river mile 365.7) to the Kansas-Missouri state line (river mile 367.2). The initial construction began in 1946. Significant improvements and repair of 1951 Flood damage followed the initial construction and were completed in 1955. The CID-MO section is designed to pass a Missouri River flow of 540,000 cfs.

The CID-MO section consists of levee, floodwalls, a levee drainage system and pumping plants, sandbag and stoplog gaps, toe and bank protection, and slope protection on the riverward slope. The floodwalls total 1.5 miles and the levee is about 430 feet in length.

3.1.3 Socioeconomic Resources

The overall existing project protects highly developed urban portions of the Kansas City metropolitan area. The protected areas encompass a major segment of the Kansas Citys' economy. Flood disruptions to this area would strongly impact the local, regional, and national economy. The Kansas City metropolitan area has a diverse and varied economic base. As a centrally located market, it is a major warehouse and distribution center and a leading agribusiness center. It ranks first in the nation as a farm distribution center and as a market for hard wheat. In addition to its agribusiness activities, the metropolitan area has major industrial activities such as auto and truck assembly, steel and metal fabrication, and food processing. The metropolitan area also fosters a growing non-manufacturing sector. Wholesale and retail industries and service organizations are now chief employers in the area.

The metropolitan area has a network of interstates and major highways that provides excellent access to each of the levee units.

- The CID Unit is accessed by means of I-70, I-35, and by I-670 which crosses over the middle of the protected area.
- The Armourdale Unit is served by U.S. 69, U.S. 169, and I-70.
- Major rail service infrastructure is present in each of the units.

Census 2010 data for 111 census tracts were compiled to describe the socioeconomic characteristics of each levee unit area as well as for the overall study area. Census 2010 data were also compiled for counties in the study area and for the Kansas City (Missouri and Kansas) Metropolitan Statistical Area (KC MSA). Although census tracts cover areas that may typically be somewhat larger than the area protected by a levee unit, the census tract data is considered to be generally representative of the protected area.

Table 3-1 displays estimates of population, employment and housing for the census tracts covering each levee unit and the study area as a whole.

Table 3-1: Estimates of Population, Employment and Housing

Unit	Population	Employment	Housing Units
Armourdale Unit	2,924	6,700	1,025
CID Unit	1,730	7,494	1,110
Study Area Total	**4,654**	**14,194**	**2,135**

Source: Census 2010

Census data, 1970 to 2000, and Mid-America Regional Council (MARC) forecasts, 2010 to 2030, for the census tracts in the study area were used to describe general trends in population, households and employment. MARC is the metropolitan planning organization for the bi-state Kansas City region. In 1970 the areas within the metropolitan flood risk management system had total population of 23,124 persons and 7,952 households. Between 1970 and 1990, the total population and number of households in the study area declined. This trend in the study area was reflective of the national trend that occurred in the 1970's and 1980's when there were population shifts to areas outside of central city areas. After 1990 the population and number of households began to stabilize and by 2000 had increased to 19,818 persons and 8,180 households in the overall system study area.

Fluctuations also occurred in the system-wide study area employment, with an overall decline from a 1970 level of 96,069 to 85,949 by 1990 and then increasing by the year 2000 to a level of 94,035. Based on MARC forecast data for the period 2000 to 2030, total employment in the system-wide study area is expected to increase steadily. Population and number of households in the area are expected to experience steady but modest growth. Figure 2 displays the general trends in population, households and employment 1970 to 2030 for the entire study area.

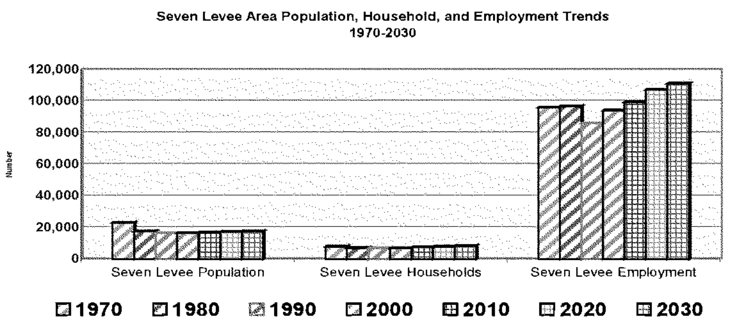


Figure 2: Kansas Citys System Study Area Population, Households, and Employment Trends

Total investment within the metropolitan system estimated at $21.9 Billion dollars (Oct 2012 price levels) and includes investment in structures, contents and equipment for commercial, industrial, residential, transportation, and public categories of investment. Depreciated replacement value for buildings and infrastructure in the study area is estimated at $8.3 billion. Businesses and residences have roughly a $13.7 billion investment in contents. Business contents include inventory, office equipment, computers, production equipment and machinery, and other miscellaneous contents. Table 3-2 shows both the Final and Interim Report study area values for comparison. Table 3-3 presents the investment breakdown for the Kansas River portion of the overall metropolitan system.

Table 3-2: Overall Investment Summary

Levee Units – Basis for Totals	Total Investment	Total Value of Structures	Total Value of Contents
Units Addressed by Final Study (Armourdale and CID)	$5,377,340,000	$2,309,040,000	$3,068,300,000
Units Addressed by Interim Study (Argentine, E Bottoms, NKC, Birmingham, Fairfax-Jersey Creek)	$16,555,600,000	$5,962,300,000	$10,593,300,000
Total System	**$21,932,940,000**	**$8,271,340,000**	**$13,661,600,000**

Note: Oct 2012 prices, rounded and shown without uncertainties

Table 3-3: Kansas River Study Area Investment for Structure and Content

Levee Unit	Number of Structures	Total Investment	Total Value of Structures	Total Value of Contents
Argentine	723	$3,053,000,000	$775,000,000	$2,278,000,000
Armourdale	1,468	$2,561,850,000	$1,241,370,000	$1,320,480,000
CID	526	$2,815,490,000	$1,067,670,000	$1,747,820,000
Total	**2,717**	**$8,430,340,000**	**$3,084,040,000**	**$5,346,300,000**

Note: Oct 2012 prices, rounded

3.2 Flood History

3.2.1 Kansas River Flood Events

Major floods on the Kansas River are usually caused by a series of short duration, high intensity storms following a prolonged period of general widespread precipitation. Table 3-4 lists the five largest annual discharges and associated stage peaks at the United States Geological Survey (USGS) gage (06889000) located on the Sardou Bridge over the Kansas River at Topeka, Kansas, river mile 83.1. The period of record for this gage is from 1904 to the present, though intermittent and anecdotal information is available from 1869. There are several gages on the Kansas River closer to Kansas City (Lawrence, Turner Bridge, 23rd Street); however, historical discharge data is not available for all locations.

Table 3-4: Flood History - Kansas River at Topeka

Year	Discharge (cfs)	Stage (ft)
July 1951	469,000	40.80
May 1903	300,000 (est.)	NA
August 1908	200,000	33.0
July 1993	170,000	34.97
June 1935	154,000	32.7

Note: Flood Stage at Topeka is 26 feet

3.2.2 Missouri River Flood Events

Floods on the Missouri River are caused by widespread storm systems over several days or weeks, sometimes combined with runoff of spring snowmelt in Wyoming, Montana, and both North and South Dakota. Table 3-5 lists the five largest annual discharges and associated stage peaks at the USGS gage on the Hannibal Bridge, just downstream of the Kansas/Missouri confluence. The period of record for stage data at this gage is from 1873 to the present; and for flow data is from 1929 to present. The highest discharge was recorded in 1951, while the highest stage peak was seen in 1993. This reflects the dynamic nature of differing flood events and the river's response to natural and man-made alterations over time.

Table 3-5: Flood History - Missouri River at Kansas City

Year	Discharge (cfs)	Stage (ft)
1951	573,000	46.2
1903	543,000 (est.)	45.0
1993	541,000	48.87
1908	402,000 (est.)	40.3
1952	400,000	40.6

Note: Flood Stage at Kansas City is 32 ft

3.2.3 Historical Flood Damages

Floods in the Missouri and Kansas River Basins are of comparatively low velocity and of several days duration. Flow data at the USGS gage on the Hannibal Bridge is available for the period 1929 to present. Before 1929 the major flood events in the Kansas Citys area occurred in 1844 (17.0 feet above flood stage), 1881 (6.8 feet above), 1903 (14.0 feet above), and 1908 (9.3 feet above). While no recorded flow information is available, the 1844 event is considered the greatest known event in the lower Missouri Basin.

In the 1903 Flood, 19 lives were lost in the Kansas Citys area, and an estimated $23,000,000 (1903 prices) in property damages was sustained. The flood of 1903 had an estimated Missouri River discharge of 543,000 cfs. The 1903 Flood gave rise to the first well-organized local efforts at major flood risk management works in the Kansas Citys area. These very old local works provided some initial line of protection layouts and features that were subsequently adapted, added to, strengthened and raised under the subsequent Federal project.

3.2.3.1 ***The 1951 Flood***

The 1951 Flood of the Kansas River exceeds all other recorded flood events at Kansas City with a discharge of 469,000 cfs on the Kansas River and 573,000 cfs on the Missouri River (measured downstream of the confluence). A two-month period of above-normal precipitation followed by intense rains over a 72-hour period in early July caused the flooding.

Beginning on Friday, July 13, 1951, a sequence of catastrophic overtopping events played out across several of the units existing at that time. Kansas River floodwaters first overtopped the Argentine Unit, then Armourdale and CID. Floodwaters eventually poured through the West Bottoms area and exited into the Missouri River by overtopping and breaching the CID-Missouri segment of levee near the old Kansas City, Missouri, Municipal Wharf. Packing plants were flooded and railroad transportation was halted due to the flooding with severe damage to tracks, rail cars, and rail yards. The flood filled the units with water depths of 15 to 30 feet. Exhibit 2 shows a photograph of the 1951 Flood along the Kansas River in Kansas City.

After devastating the three Kansas River units, the floods then threatened the intact levees located opposite and upstream of the Kansas and Missouri River confluence area. Floodwaters eventually breached a section of the Fairfax-Jersey Creek Unit near the Kansas City, Kansas, municipal wharf and flowed into the Fairfax Business District the following morning. At the peak of the flood, the Kansas River stretched from the Armourdale bluff to the Argentine bluff, with very few structures reaching above the floodwater. Of the five levee districts near the Kansas and Missouri Rivers confluence, only North Kansas City was completely saved.

Altogether about 11 square miles were flooded in the metropolitan Kansas Citys area. At least 5 persons died, and about 15,000 people were evacuated. Many residents were left homeless and five deaths in the Kansas City area are attributed to the flood. The flood caused a reported $870 million in damages throughout the Kansas River basin, and $462 million just in the Kansas City urban area (1951 price level); in 2014 prices $12.3 billion and $8.4 billion, respectively. This event is the current Kansas River flood of record for the Kansas City area.

3.2.3.2 ***The 1993 Flood***

The 1993 Missouri River flood event crested at 48.9 feet (Hannibal Bridge gage reading) on July 27, 1993, with a Missouri River discharge of 543,000 cfs. Although this discharge was less than the 1951 flood (peak 573,000 cfs), the 1993 crest of 48.9 feet exceeded the 1951 crest stage of 46.2 feet. This is likely due to changes in the river channel in the intervening years and different dynamics of Kansas vs. Missouri river flooding, upstream levee breaches, etc. All the levees in the Kansas Citys project held, although some units saw floodwaters near the top of levees, and underseepage problems were evident in several units. Several of the levees sustained some damages (erosion, etc.) and were subsequently repaired.

This event was a Missouri River event with no coincident flooding on the Kansas River. However, there was still loading on the Kansas River Units caused by backwater effects from the Missouri River, starting near full height at the confluence and decreasing upstream. In two reaches of the CID Unit near the confluence, sand boils and seeps were observed in two locations areas where floodwaters were measured at two and three feet below the top of the levee. At the upstream end of the CID Unit, floodwaters were measured at seven feet below the top of the floodwall, and one sand boil was observed. In the Argentine Unit, the furthest upstream unit from the confluence, flood waters reached within nine feet of the top of the levee and several sand boils were observed. At one point during the event, the very downstream end of the Armourdale Unit was forecasted for overtopping. Sandbags, concrete barriers, and steel beams were used to raise the levee and floodwall in this reach of the unit approximately two feet, but the peak stage stabilized at 1.5 feet below the top of the existing levee at this location and did not reach into the additions. By comparison, at the upstream end of the Armourdale Unit floodwaters remained approximately ten feet below the top of the levee. Despite these issues, and the flood fight efforts that were required, the 1993 event is not considered a full capacity test of the Kansas River units. Had floodwaters been higher in the more upstream reaches of these units and the loading more evenly distributed long the length of these units, as would be expected to occur in a Kansas River flood event, the observed underseepage concerns would have been worse and the potential for structural failures much greater.

An estimated $4.57 billion (1993-1994 price level) in damages were prevented by the Kansas Citys flood risk management project (The Great Flood of 1993, Post-Flood Report, U.S. Army Corps of Engineers, September 1994). Even though none of the levees in the Kansas Citys project experienced overtopping, and observed underseepage was effectively managed, the combination of relief well flows, pump stations operating near or at capacity, and local tributary flooding, resulted in interior flood damages within the protected area.. Damages to Kansas City, Kansas, utilities reached several million dollars. Kansas City, Missouri, reported more than $15 million in damage to public infrastructure. Kemper Arena and the American Royal Buildings within the CID Unit suffered about $2.5 million in water damage to flooring and electrical circuits. The downtown airport sustained damages of nearly $3 million, and pollution control and public works facilities sustained an estimated $8 million in damage. The 1993 Flood is the Missouri River flood of record for the Kansas City area. Exhibit 3 shows a photo of the Missouri and Kansas Rivers confluence during the 1993 flood and the flood event hydrograph for the Kansas City gauge on the Missouri River.

3.2.3.3 ***Recent Flood Events***

Several flood events occurred during the course of this study that were significant to the Missouri River Basin, even though they did not directly impact the Kansas Citys System. Events in 2007, 2010, and 2011 loaded levee systems and caused overtopping breaches both up and downstream of Kansas City, but did not create significant concern locally. The peak discharges for these three Missouri River events at Kansas City were 286,000 cfs, 212,000 cfs, and 245,000 cfs, respectively. The 2011 event is particularly notable due to the prolonged duration of the event, 145 days; a result of record discharges from the upstream basin reservoir system. It should be noted that the upstream levee breaches that occurred during these events most likely lowered the river stages at Kansas City and contributed to these events not directly impacting the local system. The 2011 Flood is currently the flood of record in the Upper Missouri basin.

3.3 Authorized Flood Risk Management

Multiple reports were prepared by various entities during the 1930's proposing different plans for projects at Kansas City. The authorized discharges of the Kansas River Units, pursuant to the Flood Control Act of 1936, are contained in the October 31, 1936 report titled "Missouri & Kansas Rivers, Kansas Citys, Flood Control Project, Project Report." This report states that the project should accommodate a probable maximum flow in the Kansas River of 370,000 cfs, and a combined flow of the Missouri and Kansas Rivers of 630,000 cfs.

The determination design discharges depended greatly on assumptions about the center of storm events. The following excerpt is taken from House Document No. 342 (78^{th} Congress; June 9, 1943): "With an excessive storm centered principally over the Kansas River basin, the design-flood discharge at the Kansas Citys would be 170,000 cfs from the Kansas River and 330,000 cfs from the upper Missouri River, or a total of 500,000 cfs. Conversely, with an excessive storm centered principally over the Missouri River basin, the design flood discharges would be 80,000 cfs from the Kansas River and 460,000 cfs from the upper Missouri River, or a total of 540,000 cfs."

The available construction plans for the levee units indicate 540,000 cfs as the design discharge for units downstream of the confluence, 460,000 cfs for Missouri River units upstream of the confluence, and 390,000 cfs for Kansas River units. The larger combined design discharge of 630,000 cfs contained in the 1936 report was apparently never adopted into the units' construction history.

Each unit was designed and constructed to successfully pass a specified river discharge with adequate freeboard, or levee height, above the estimated water surface elevation. Discharge and level of performance is a complex issue for this system due to the confluence of the Kansas River with the Missouri River occurring within the study area, and given that each river has an independent runoff basin. Additional details relating to design hydraulics are provided later in this report.

Another complicating issue when discussing flood discharges and probabilities is changes to the preferred terminology. Expressing discharge probability in percent chance exceedance (occurrence) is currently preferred by the Corps of Engineers in lieu of a flood return interval expressed in years. The terms "flood return interval" and "level of protection" have been in use for many years and are familiar to the general public. However, over the years it is apparent that misconceptions have developed around the return interval nomenclature, such as, the expectation that the 100-year flood can only occur once in a period of 100-years. Percent chance exceedance is a more statistically accurate expression of the probability of a specific flood discharge occurring in any given year. For example, the flood magnitude with a 1 in 100 probability of occurrence in any given year is designated as the 1%-chance flood event.

The phrase "level of protection" can also be misunderstood to indicate that only floods above a certain magnitude are capable of causing levee failure or damages, when in fact the dynamic and differing conditions of separate flood events will impact the existing levee in different ways. From one flood to the next, the same measured discharge flow will rarely produce the same water surface elevation. It is now more common to express water surface elevations in terms of probability, or confidence limits, to establish a range within the flood elevation can be expected to occur for a given discharge. This, in turn, allows for the inclusion of risk and uncertainty in the evaluation and expression of probable levee performance under different conditions.
To account for the uncertainties inherent in calculations of flood probabilities, Corps of Engineers risk and uncertainty (R&U) analytical tools and procedures were used in this feasibility analysis, as per ER 1105-2-101 "Risk Analysis for Flood Damage Reduction Studies". It should be noted that the risk analysis and evaluations resulting from this type of analysis are not directly comparable to the discharge-plus-freeboard performance criteria used for the original authorized levee design. The use of level of protection is maintained when referencing and quoting historical documents and sources that relied on these terms.

3.4 Construction History and Design Discharge

Table 3-6 provides a summary of the major periods of construction and the current design discharge conveyance targets for each of the units in the project.

Table 3-6: Summary of Levee Unit Construction History and Design Discharge

Levee Unit	Initial Federal Project Completed (year)	Last Federal Modification (year)	River	Design Discharge (cfs)
Armourdale	1951	1976	Kansas	370,000[1]
CID, Kansas	1948	1979	Kansas	370,000[1]
CID, Missouri	1947	1955	Missouri	540,000[2]

[1] 1936 Project Report
[2] 1943 Congressional Documentation

After the catastrophic 1951 Flood, the Kansas River levee units were reauthorized to pass higher design discharges. Table 3-7 shows the increased design discharges along with coincident Missouri River discharges. However, the Missouri River levees downstream of the confluence were not improved as a result of the 1951 Flood event, even though the 1951 Flood discharge exceeded the original design discharge of these units.

Table 3-7: Revised Design Discharges for the Kansas River Levees ("1962 Mod")

Levee Unit	Kansas River Authorized Design Discharge (cfs)	Missouri River Coincident Discharge (cfs) Upstream of Kansas River	Missouri River Coincident Discharge (cfs) Downstream of Kansas River
Armourdale	390,000	220,000	610,000
CID (Kansas)	390,000	220,000	610,000
Argentine	390,000	220,000	610,000

In general, the "1962 Mod" discharges were used to develop higher design water surface profiles for levee raises in the affected units. The final elevation of the levee was determined by taking the design water surface profile and adding freeboard. The levee units were authorized to pass specified discharges on the Kansas and Missouri Rivers with either 2 or 3 feet of freeboard. The other units along the Missouri River have a design level of performance as authorized in 1944. Subsequently, the Liberty Bend Cutoff was constructed along the Missouri River in the 1950's and aided in overall conveyance of flood discharges through the Kansas Citys reach.

3.5 Current O&M Requirements

The individual units of the Kansas Citys flood risk management system were turned over to the levee unit sponsors following each construction effort. Operation, Maintenance, Repair, Rehabilitation, and Replacement (OMRR&R) of the units and features is accomplished by the respective sponsors and annually inspected by the Kansas City District. The primary responsibilities for sponsors of Federal flood risk management projects are detailed in the Code of Federal Regulations (CFR) Title 33 - Navigation and Navigable Waters, Chapter II - Corps of Engineers, Department of the Army, Part 208 - Flood Control Regulations, Maintenance and Operation of Flood Control Works. Also providing guidelines regarding operations and maintenance requirements is Engineering Regulation (ER) 1130-2-530 (Project Operation).The Operation and Maintenance Manual for each levee unit addresses project specific sponsor responsibilities and contains the full text of Title 33. The sponsors all have operating staff that are familiar with the details of effective maintenance practices. Each sponsor maintains their own office and legal records, and operation and maintenance records to the extent they determine useful. The Corps of Engineers does not normally inspect nor duplicate these records.

Each unit is inspected annually and a more in-depth Periodic Inspection is conducted every five years. The sponsors of each unit have continued to adequately and effectively fulfill their O&M responsibilities since project construction, as documented by Kansas City District inspection records. Any deficiencies or encroachments on the units identified in inspection reports have generally been minor in nature, not significantly impacting project operations or readiness, and are being addressed by the sponsor in a timely manner. Sponsor operations and maintenance is an important and indispensible component of ensuring the existing system provides the intended risk management benefits.

3.6 Existing Reservoir System Effects

3.6.1 Effects of Kansas River Basin Reservoir System

A multi-purpose system of reservoirs in the Kansas River basin was authorized in the Flood Control Act of 1944. Eighteen (18) Federal lakes/reservoirs now exist in the Kansas River basin; seven managed by the Corps of Engineers and eleven by the Bureau of Reclamation. The seven Corps lakes are large enough and close enough to the Kansas City area to have a major effect on flows passing through the Kansas City system.

This system was authorized, in part, to act in concert with the system of Federal levees in Kansas City and other areas to reduce flood damages in the areas protected by the levees (the levees in the Kansas City area had been previously authorized). Modifications to this original 1944 lakes authorization have appeared in subsequent Flood Control Acts, but the basic objective of providing a coordinated flood risk management system on the Kansas River, as outlined in the 1944 Act, has been preserved. The Kansas City District operates these reservoirs in compliance with the original intent of that Act.

The upstream reservoirs in the Kansas Basin are operated with consideration of Kansas and Missouri River flows. Depending on the amount of water stored in their flood control zones, each reservoir restricts releases based on downstream conditions. Reservoir releases will not increase downstream flow more than the limits presented in Table 3-8 at the Desoto gage on the Kansas River, the Kansas City gage on the Missouri River, or the Waverly gage on the Missouri River.

Table 3-8 Kansas River Basin Reservoirs Releases: Downstream Flow Limits

	Desoto Gage Kansas River	Kansas City Gage Missouri River	Waverly Gage Missouri River
Phase 1: Lower zone of flood control pool	66,000 cfs	176,000 cfs	90,000 cfs
Phase 2: Middle zone of flood control pool	110,000 cfs	220,000 cfs	130,000 cfs
Phase 3: Upper zone of flood control pool	130,000 cfs	240,000 cfs	180,000 cfs

Cfs = cubic feet per second

3.6.2 Effects of Missouri River Basin Reservoir System

There are six major Federal lakes/reservoirs on the main stem of the Missouri River in the Dakotas and Montana. The reservoir furthest downstream is Gavins Point in southern South Dakota, which is about 440 river miles upstream of the Kansas City area. This system of reservoirs provides flood risk management benefits all along the Missouri River, but the system does not operate specifically for the Kansas City area. Any release at Gavins Point undergoes a five day travel lag before arrival of that water at Kansas City. The Kansas River levee units can be indirectly impacted by Missouri River reservoir operations when considering Missouri River backwater effects and the possibility of coincident flooding scenarios.

3.7 Recent Evaluations of River Flow Frequency

Following the flood of July 1993, the Corps of Engineers undertook a major reevaluation of the flow frequency of the upper Mississippi, Missouri and lower Illinois Rivers. The resulting Upper Mississippi River System Flow Frequency Study (UMRFFS) constituted an update of the previous flow frequency estimates then in use for these rivers. On the Missouri River, the previous flow estimates were completed and published in 1962. The UMRFFS study provided revised flow frequency estimates and flood profiles.

To fully evaluate the operations of the Kansas River basin reservoir system as part of the UMRFFS study, updated flow information and flow frequency estimates were also generated for the Kansas River from its mouth to Manhattan, Kansas. Table 3-9 summarizes the regulated flow frequency estimates as applicable to the Kansas Citys study[1]. The Kansas River data from Table 3-9 is presented graphically in Figure 3.

Using the data and information from the UMRFFS study, and other studies, the Corps of Engineers conducted a complete hydraulic and hydrologic analysis, including development of an existing conditions HEC-RAS (River Analysis System) model calibrated to the flood event of 1993 for use in studying this system. This analysis also addressed the effects of potential improvements for all seven units of the existing system and is the basis for the system EIS. The results and findings of this system wide hydraulic analysis were completed and published in the Engineering Appendix to the 2006 Interim Feasibility Report. The hydraulic information detailed herein is taken from the previously completed analysis; no new analysis was conducted for the purposes of this Final Feasibility Report as the previous data was determined not to have changed. For reference, the previous Hydrology and Hydraulics chapter is repeated in the Engineering Appendix accompanying this report.

Table 3-9: Study Area Flow Frequency Data

Annual Percent Chance of Exceedance	Missouri River Downstream of Blue River (cfs)	Missouri River Downstream of Kansas River (cfs)	Missouri River Upstream of Kansas River (cfs)	Kansas River at Mouth (cfs)
0.2	537,000	530,000	358,000	341,000
0.5	459,000	454,000	316,000	283,000
1	405,000	401,000	287,000	241,000
2	354,000	351,000	257,000	202,000
5	292,000	289,000	220,000	150,000
10	247,000	245,000	192,000	121,000
20	203,000	201,000	162,000	90,700
50	143,000	142,000	120,000	51,200
80	104,000	103,000	89,500	26,400
90	89,100	88,300	77,200	18,700
95	78,800	78,100	68,500	14,000
99	63,400	62,900	55,100	8,200

[1] Sources: Upper Mississippi River System Flow Frequency Study, 2001; and the Kansas River Hydrology Report, 2002

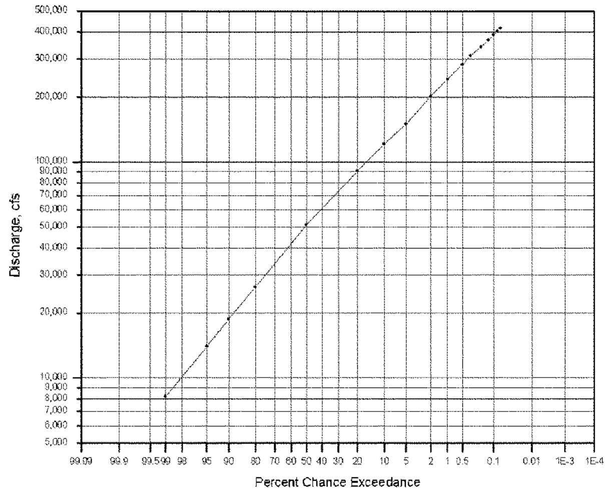


Figure 3: Discharge-Frequency Curve – Kansas River at Mouth

Since flood events above the 0.2% chance of exceedance (500-year) event needed to be considered in this study, the discharge-frequency curves were extended up to the 0.067% chance of exceedance (1,500-year) flood event. Table 3-10 summarizes all of the discharges developed for use in this study.

Table 3-10: Summary of Flood Discharges Used in this Study

Frequency in Percent Chance of Exceedance	Missouri River Downstream of Blue River (cfs)	Missouri River Downstream of Kansas River (cfs)	Missouri River Upstream of Kansas River (cfs)	Kansas River at Mouth (cfs)
0.067%	637,000	625,000	414,000	417,000
0.080%	621,000	610,000	403,000	403,000
0.100%	600,000	590,000	390,000	388,000
0.133%	573,000	565,000	377,000	367,000
0.200%	537,000	530,000	358,000	341,000
0.500%	459,000	454,000	316,000	283,000
1.000%	405,000	401,000	287,000	241,000
10.000%	247,000	245,000	192,000	121,200

The Discharge-Frequency relationships summarized in Table 3-10 indicate that the authorized design discharge of 390,000 cfs has an annual exceedance probability of less than 0.1%. Given the very low chance of occurrence of a flood of this magnitude it was determined to be neither practical nor economical to evaluate existing performance, or develop modification alternatives, for the authorized discharge profile. For consistency with the desired benefits and uniformity of risk management within the levee system, evaluations of current and future project performance conducted for this study focused on the 0.2% chance flood.

It is of interest to explore and understand the possible reasons why the existing units are unable to perform at the discharge for which they were originally designed and constructed. The Kansas River Unit raises of the 1970's were based on hydraulics utilizing 1951 cross-sections. These cross-sections were taken after the July 1951 flood event and they show that in the lower reach of the Kansa River major channel scour occurred during the flood event. The major assumption for using the 1951 cross-sections for hydraulic analyses and levee design profile is taken from General Design Memorandum #1 and is as follows: "It was determined that for design conditions, the cross-sections that existed on the Kansas River shortly after the 1951 flood would represent the channel cross-section during passage of the design discharge."

The Kansas River Unit raises of the 1970's were also based on hydraulics utilizing roughness coefficients (Manning's n-values) of 0.025 and 0.05 for the channel and overbank areas, respectively. Aerial photos of the lower Kansas River shortly after the 1951 flood event indicate very little obstructive vegetation in the overbank areas. This is more evident from about river mile 4 upstream to river mile 10. From levee to levee, a relatively clean conveyance area for carrying flood flows existed after the 1951 event.

The hydraulic model developed for this Kansas Citys Levees Feasibility Study is based on mapping and aerial photography obtained between 1995 and 1999 (most currently available data at the time of model development). In the last 40-50 years since the 1951 flood, the lower reach has accreted with sediment and the composite roughness values across the entire cross-section have increased dramatically. These factors have lead to higher water surface elevations than those calculated for the 1970's Kansas River Unit raises. It should be noted, that accretion has significantly slowed and vegetation growth appears to have reached its maximum density, therefore these factors are unlikely to significantly affect flood stages in the future.

3.8 Assessment of Existing Levee Integrity

During early portions of the existing conditions assessment, the O&M Manuals and Record Drawings were reviewed and were followed by field visits with sponsor representatives to compare available survey information with actual field conditions.

In 2001, a centerline survey of the top of levee was conducted for verification of the O&M Manual elevations and was used in conjunction with the hydrologic and hydraulic analyses. A review of the centerline survey indicated that some areas along the levee were slightly lower than shown in the O&M Manual. Based on this, a resurvey of portions of the centerline was conducted in late 2003. The results of the resurvey confirmed that some areas were slightly lower. Review of the levee condition found no indicators that these low spots were due to post construction settlement or geotechnical foundation concerns. This information led to discussions with sponsors and additional emphasis on preparations for emergency flood fighting (sandbagging) or local maintenance and repair of the low spots.

The study assessments provide insight into both the existing levee performance and the economic damages expected under existing conditions for an array of high water events. Risk and uncertainty analysis results and observations of levee performance during flood events form the basis for the identification of opportunities for risk reduction measures. The critical reaches for geotechnical underseepage failure and slope stability risks were identified and analyzed in each unit. Critical reaches for geotechnical risk are determined by several factors including levee height, slope, and soil type, and are the locations where underseepage or stability risks are expected to be the highest. The structural components of each unit were analyzed and compared to the current minimum factor of safety (FS) for hydraulic uplift, strength, and stability. Features that did not meet the minimum required factor of safety were further evaluated to determine probability of failure (PoF) with water at the top of the levee (TOL). .

Probability of failure curves (probability vs. water surface elevation) were prepared for the most critical features and combined in the economic analysis to determine an overall probability of failure for each unit. The details of this analysis, which are based on evaluations using the HEC-FDA program, are presented in Appendix B. The overall existing condition engineering performance is shown in Table 3-11. The locations and features not meeting the factor of safety and showing the highest probabilities of failure in each unit are summarized in Table 3-12.

In Table 3-11, the "Conditional Exceedance Probability – Overtop or Breach" represents the probability of levee unit failure from all possible failure modes (overtopping, geotechnical, and structural). As shown, the current existing failure risk is significantly high in both units: 3.5% annual probability for the Armourdale Unit and 0.3% for the CID Unit. The "Conditional Exceedance Probability – Overtopping Only" represents that portion of the existing failure probability attributable to overtopping failure only.

Table 3-11: Engineering Performance (Existing Conditions)

	Armourdale	CID
Annual Exceedance Probability* (median)	3.5%	0.33%
Annual Exceedance Probability* (expected)	3.7%	0.47%
Long Term Risk (chance of exceedance during indicated period)		
over 10 years	31.4%	4.6%
over 30 years	67.7%	13.2%
over 50 years	84.8%	21.0%
Conditional Exceedance Probability - Overtop or Breach**		
10.0% event	16.4%	0.0%
4.0% event	22.2%	0.0%
2.0% event	31.7%	1.4%
1.0% event	54.5%	11.3%
0.4% event	81.4%	42.4%
0.2% event	91.9%	66.6%
Conditional Exceedance Probability - Overtopping Only		
10.0% event	0.0%	0.03%
4.0% event	0.0%	0.03%
2.0% event	0.6%	0.5%
1.0% event	7.9%	6.6%
0.4% event	36.7%	33.5%
0.2% event	61.4%	58.8%

*Annual exceedance probability is the chance of experiencing any flood event - of whatever magnitude - within any year. Expected values include Monte Carlo risk and uncertainty modeling.
**Conditional exceedauce probability is the probability that specified flood event would overtop or breach the levee.

Table 3-12 Summary of Existing Conditions in Areas of Concern

ARMOURDALE UNIT				
LOCATION	**DESCRIPTION**	**FS NOT MET**	**PoF**	**CONSEQUENCE**
Sta. 185+70	5th Street Pump Station	Uplift	100%	Unit will flood
Sta. 129+20	12st Street Pump Station	Uplift/Strength	100%	Uplift: Unit will flood
Sta. 156+75	Mill Street Pump Station	Uplift/Strength	100%	Strength: Post-flood repair
Sta. 222+00	Slope Stability Critical Location	Slope Stability	24%	Unit will flood
Sta. 276+00	Underseepage Critical Location	Underseepage	8%	Unit will flood
CENTRAL INDUSTRIAL DISTRICT UNIT - KANSAS SECTION				
LOCATION	**DESCRIPTION**	**FS NOT MET**	**PoF**	**CONSEQUENCE**
Sta. 83+52	Ohio Street Pump Station	Strength	100%	Post-flood repair
Sta. 132+20	Closure Structure	Strength	99%	Unit will flood
Sta. 166+31	Closure Structure	Stability	20%	Unit will flood
Sta. 104+51	Closure Structure	Stability	6%	Unit will flood
Sta. 85+00	Underseepage Critical Location	Underseepage	4.5%	Unit will flood
CENTRAL INDUSTRIAL DISTRICT UNIT - MISSOURI SECTION				
LOCATION	**DESCRIPTION**	**FS NOT MET**	**PoF**	**CONSEQUENCE**
Sta. 19+39 to 22+31	Floodwall	Sliding Stability	100%	Unit will flood
Sta. 63+15	Closure Structure	Foundation Stability	64%	Unit will flood
Sta. 0+00 to 3+49	Floodwall	Strength	14%	Unit will flood

Both the Armourdale and CID units have some probability of breach under existing conditions, but the probability of breach is much greater for the Armourdale Unit. For both units, the analysis indicates that the unit will structurally fail before overtopping. Deficiencies in the Mill St. Pump Station, the 12th St Pump Station, and the 5th Ave Pump Station are the major contributors to the existing condition probability of geotechnical/structural failure, which could cause a breach in the Armourdale Unit at flood levels below TOL. For the CID Unit, it is structural gatewells, floodwalls, and stoplog gaps that contribute to a lesser, but still significant, probability of structural/geotechnical failure and breach for events below TOL.

If all geotechnical and structural failure risks listed in Table 3-12 were addressed, a significant overtopping risk would still remain for the target 0.2% chance flood event. These findings for overtopping risk in the lower Kansas River show that these units do not reliably achieve the authorized 390,000 cfs conveyance target. Similar information was presented in the Interim Report on the Argentine Unit. This indicates the need for a general increase in the existing overtopping protection along the lower Kansas River.

The details of the engineering performance analyses of geotechnical and structural features of the Armourdale and CID Units, including floodwalls, drainage structures, closure structures, and pump stations, are provided within the appropriate chapters of Appendix A to this report.

3.9 Future Without Project Scenario

The without condition scenario is a narrative description of the significant water and related land resources conditions and their impacts that could exist if the planning partnership takes no action. In best practice all scenarios are developed after careful consideration of what is reasonably known and not known about the future. When most of the alternative futures are relatively similar, differing only in the details, some of which may be significant, it is both possible and desirable to use a single most likely without condition scenario. Uncertainties in such a scenario can be explored using sensitivity analysis and other risk-based analytical techniques within the framework of that scenario. When uncertainties are so great as to produce significantly different future scenarios it is not reasonable to single out one scenario as most likely. In these instances scenario planning with multiple without condition scenarios may be necessary.

3.9.1 Socioeconomic Considerations

The Armourdale and Central Industrial District Units last experienced catastrophic flooding in July of 1951. Following this devastating flood these areas struggled for years, even decades, to return to their pre-flood economic vitality. The meat packing industry that thrived in these areas never fully recovered (the Armourdale area took its name from the Armour & Co. plant). One plant did not reopen and the rest were gone within twenty years. The impact of the flood was not the only factor to affect the meat packing industry, but certainly it was a significant one. The famous Kansas City stockyards began a slow decline with the departure of the meat packing industry, finally closing down for good in 1991. The residential populations within these areas also dropped significantly in the years after the flood as jobs left and people moved out of the flood prone areas. These are just a few examples of the impacts that severe flooding has on affected areas, and ones that could easily be repeated in the future.

Since the 1951 flood, the industries that remained have recovered and new industries have arrived. Convention facilities, restaurants, artist studios, commercial offices, and other uses have slowly moved into the study area. In keeping with this trend, only gradual, minor changes in population, employment, and land use are expected within the study area. The population of the Kansas City metropolitan area has been relatively stable according to the 1980 through 2010 census. Significant changes in population and land use in relation to existing conditions are not expected. However, several important planned commercial and residential developments have been identified in certain areas during discussions with sponsors and occupants of the study area. In addition, several road and highway improvement projects have been proposed, or are being implemented currently, that will increase access and traffic flow to, and within, the study area. These developments are expected to add to the general overall economic activity.

Opportunities for new development in the future are limited by the dense urbanization already existing and the scarcity of available open ground. Redevelopment efforts, or other changes from the current land use, may be restricted by floodplain zoning and flood insurance requirements. Most of this area would be within the base flood plain if not for the current flood risk management project. Any development along the river outside the line of protection would be precluded by the regulatory floodway which covers the entire span between the left- and right-bank levees.

While the identified trends and assumptions indicate that the existing socioeconomic fabric of the study area will remain relatively the same, and may improve some, the relative risk of a damaging flood increases into the future. Should another catastrophic flood occur with the study area, economic stability would be severely impacted, as has been seen before. It is reasonable to assume that some businesses and residents impacted by such a flood would not return to, or rebuild within, the study area. Large regional and national businesses currently in the study area may choose to relocate jobs completely outside of the Kansas City area, causing significant regional economic impacts.

3.9.2 Hydrologic and Hydraulic Considerations

3.9.2.1 *General*

The Hydrologic Engineering Center River Analysis System (HEC-RAS) computer model is used to calculate the probable stage-discharge relationship at a selected future date based on the best available current data, the incorporation of any known projects planned to be completed within the study area, and any long term natural river processes that may affect future stages. In the development of the future without scenario, it is important to adequately detail and validate the current data and future assumptions that are input into the model.

3.9.2.2 *Expected Future Condition Changes*
A critical assumption in the future conditions analysis is that hydrologic conditions along the Missouri River and the Kansas River are relatively static and that flows used in the existing conditions study generally apply to the future conditions analysis. This assumption was also used in the development of the *Upper Mississippi River System Flow Frequency Study* (UMRFFS), 2003, which was based on the study of 100 years of gage records along the Mississippi River and tributaries, including the Missouri and Kansas Rivers. The UMRFFS superseded the previous Missouri River hydrology published in 1962 in the report titled *Missouri River Agricultural Levee Restudy Program*. The newly published flows in the UMRFFS were used in this study for both present and future conditions.

The future condition scenario does not anticipate the construction of any major Federal or local projects along the Kansas River that will have the capacity to affect the water surface elevations in the future. However, some of the natural processes occurring on the Kansas River are similar to processes occurring on the Missouri River.

Examination of aerial photography sequences show significant tree growth on certain lower Kansas River foreshore areas during the years from 1955 to the 1990's, especially on the left and right foreshores from the upper limits of the studied reach to approximately Kansas River mile 3.5, a distance of approximately 6.5 miles. Downstream of river mile 3.5, very little vegetation exists on the foreshore. Some accretion is noted along the studied reach, but not to the extent of the Missouri River. This difference may be due to the absence of navigation structures in the Kansas River.

The future without scenario assumes that because the upper reach is heavily vegetated for the existing conditions, the future conditions will not be worsened by further maturity of these growths. Based on a review of the vegetation patterns from 1955 to the present, it is also assumed that the amount and extent of vegetation on the lower reach from river mile 3.5 to the mouth will remain relatively stable. Therefore, the future natural condition along the Kansas River does not change from the existing condition.

3.9.2.3 *Missouri River Degradation*
The Missouri River between miles 340 and 400 in the Kansas City reach has exhibited down cutting of the river bed. This phenomenon has been observed by evaluation of Missouri River gage data collected over a long period of time. As the bed of the Missouri lowers, degradation begins to travel upstream many of its tributaries, including the Kansas River. Bed degradation can have many negative impacts to infrastructure such and bridges and water supply intakes, and can impact the riverward stability of existing flood risk management features.

The potential causes of degradation, documentation of its effects, and potential alternatives for management or mitigation are currently being evaluated under separate study efforts. This Kansas Citys feasibility study is directed only towards the analysis of levee unit performance under flood conditions. Channel degradation has been considered where it has demonstrable potential effects on flood risk management performance. However it was determined unnecessary to project future degradation changes into future without project for the Kansas Citys study. The results and recommendations of the separate degradation study efforts will be reviewed when available and incorporated into future project design efforts where deemed necessary.

3.9.3 Period of Analysis and Related Assumptions

Both the future with and without condition scenarios are evaluated over a 50 year period of analysis to allow a consistent and appropriate comparison of alternatives. The period of analysis is the time horizon for which project benefits and project operation, maintenance, repair, rehabilitation and replacement (OMRR&R) costs are evaluated. The period of analysis begins with the base year condition (considering resources in the study area and economic and engineering factors) thought to exist in the first year a project alternative is expected to become operational. Engineering and economic data is also developed (projected) for a future year about 20 to 30 years out from the base year. The analysis years used in this Final feasibility study are 2026 for the base year and 2049 for the future year, with the total 50 year period of analysis ending in 2076.

In this study, certain assumptions related to the period of analysis were made:

- River stage uncertainty values were increased from 1.5 ft. to 1.8 ft. in the future year 2049; this reflects the increased difficulty in predicting stages far in the future.
- No significant increase in economic development is projected for the 50 year period of analysis as much of the protected area is essentially built-out.
- Beyond the future condition year of 2049, the expected annual damage is assumed to be constant in the remaining years of the period of analysis.

These assumptions provide the framework of the future without scenario in which the analysis of future flooding impacts is conducted. The expected annual damage for each year in the period of analysis is then computed, discounted back to present value and annualized to determine the equivalent annual damage for any year during the analysis period.

3.9.4 Without Project Scenario Conclusion

If modifications and improvements to the existing flood risk management system are not implemented through a Federal cost-shared project, the non-Federal Sponsors will be faced with either a significant financial burden of trying to implement the project themselves, or they will have to rely on flood-fighting to protect the study area from future floods. Neither option alleviates the existing flood risks or the increasing risks as the exiting project continues to age.

The trends and assumptions discussed in this section establish a future scenario in which the without-project and with-project conditions and flooding impacts can be analyzed and compared. The specific details and results of these analyses are discussed later in this report.

4 Development of Alternatives

This chapter presents the results of steps three through six of the planning process, the formulation, evaluation, and comparing of alternative plans leading to plan selection. These steps are difficult to separate into discrete activities as the evaluation and comparison of management measures and alternative plans often leads to reformulation, additional evaluation, and continued refinement, before a final recommendation is reached.

Early problem definition efforts required that the study establish the existing performance condition and future without project condition scenario for the individual units in the study area. The primary means of quantification of these baseline conditions was through the development of risk and reliability metrics (for flood condition performance) by using risk and uncertainty (R&U) principles and the Corps of Engineers HEC-FDA program. This is significant due to the numerous elements and features of the units which required the identification and quantification of performance weaknesses.

Much of the analysis used data and observations from recent high water events, especially those in 1993 and, to a lesser degree, 2011. This updated engineering analysis, along with the economic existing conditions analysis, establishes a complete R&U approach to estimating existing conditions flood damages. The engineering and economic evaluations were taken together with a summary baseline environmental review and an HTRW review to develop the existing conditions.

The initial broad feasibility evaluations of existing conditions undertaken during the first two to three years of this study allowed subsequent formulation efforts more focus. The development of measures to increase reliability was narrowed to the candidate sites which offered the best opportunity for significant reliability improvements and potential economic return on investment. These candidates were also reviewed for compatibility with the basic planning objective which emphasized the desirability of a relatively uniform level of flood risk management across the metropolitan system.

As feasibility progressed, the development of reliability improvements were thus focused on those specific areas identified as having relatively low reliability; areas where low reliability significantly compromised the projects original intended level of performance. Engineered reliability remedies and improvements were developed considering both the improvements to individual unit performance and the performance of the whole system.

Alternative plans shall be formulated to identify specific ways to achieve planning objectives within constraints so as to solve problems and realize the opportunities identified. An alternative plan consists of a system of structural and/or nonstructural measures, strategies, or programs that meets, fully or partially, the planning objectives. The first phase in plan formulation is the identification of management measures, followed by combing the measures into plans as appropriate.

The results of the existing conditions analysis, and observations and effects from historic and recent flood events, were used to identify and formulate potential solutions aimed at lowering the risk of flooding for units under study.

At times additional measures and alternatives surfaced leading to formulation of new plans or plan reformulation. As the alternatives passed through subsequent evaluation and screening processes, the economic analysis of each alternative was used as a primary ranking factor in the final selection. Having passed review for engineering adequacy, environmental and public acceptability, and other evaluation criteria as described below, the remaining alternative with the highest net benefits to the national economy was identified as a component of the overall recommended plan.

Note regarding Price Level: Throughout the planning and alternative development process, cost estimates and economic analyses were prepared at various times to assist in decision making regarding the efficiency and performance of measures and alternatives. Due to inflation and interest rate changes, cost estimates and economic benefits calculated in different years are not comparable. Cost and economic information presented in this report is shown as calculated at the time it was used for decision making, and is labeled with the appropriate Price Level. Price Level is designated by the first month of the Fiscal Year in which the prices were effective, for example an October 2005 Price Level is Fiscal Year 2006, and so on. The final Recommendation of this report presents the recommended project costs and benefits in the appropriate Price Level for the Fiscal Year in which this report is published and approved.

4.1 Plan Formulation Rationale

Planning studies are required to examine and address the Federal criteria of completeness, efficiency, effectiveness, and acceptability. Alternatives and recommendations are also closely examined for their potential to impact the environment. To adequately address these criteria, the development and screening of alternatives should consider of a number of evaluation factors. Primary among those factors are the following:

- Engineering adequacy of the proposed solutions (effectiveness)
- Contribution to planning objectives (completeness of the solution)
- Consistency with planning constraints and authorities
- Environmental, cultural, and public acceptability
- Early cost indicators (early efficiency indicators for screening purposes)
- Induced damages considerations (where applicable)
- Hazardous and regulated waste site constraints (where applicable)
- Constructability (are construction techniques and quality difficult to attain at reasonable price)
- Construction site constraints (given existing features and development)

Engineering Adequacy: The engineering adequacy of alternatives was analyzed and reviewed during the initial screening process. Any alternatives which could not meet the minimum technical criteria for the expected flood conditions were eliminated from further review. This is a key effectiveness criterion and normally must be met. The amount of engineering analysis necessary to perform the engineering review was generally considerable and is contained in the various Engineering Appendices.

Environmental Acceptability: Environmental acceptability of alternatives was reviewed in concert with appropriate resource agency guidance. Any alternative which had major disruptive effects on the environment was normally screened out. A typical formulation exercise would involve adjusting some of the alternative measures so as to minimize any environmental impacts when such impacts could not reasonably be avoided.

Cultural Acceptability: Any cultural resources present were considered as the areas likely to be affected by a solution were determined. Steps were taken during the alternatives screening and refinement process to generally avoid any impacts to culturally significant sites.

Early Cost Indicators (efficiency): Early approximate cost indicators related to the various alternatives were used to determine if an alternative was prudent for further examination. As the evaluation process continued, cost estimates and economics were refined. The detailed cost estimating and economic analysis normally focused only on those alternatives that remained viable solutions after early screening criteria were passed.

Induced Damages: While this consideration is similar in some respects to the floodway conveyance factor, the analysis actually goes one step further and addresses the possibility of induced impacts during extremely rare events in which the order of overtopping may be altered by levee raise proposals.

4.2 Key Uncertainties

A number of preliminary uncertainties were identified and investigated as thoroughly as possible during the study. These uncertainties were important considerations in selection and evaluation of effective management measures. These included the following:

- Impact of Missouri River bed degradation on the Kansas River. A separate Corps of Engineers study is currently underway examining bed degradation in the Missouri River. As the Missouri river bed scours itself lower, there is potential for this degradation to begin travelling up tributary streams, such as the Kansas, and alter future water surface profiles, undercut river bank slopes, etc. At this time, the study has not reached any firm conclusions or recommendations to slow or reverse the degradation, but that is part of the study's goals and objectives. For the purposes of evaluating future flood risk management on the Kansas River, it is assumed that some future measures will be in place to address degradation, and that the future water surface profile models used for establishing new levee heights will not be significantly impacted. The future findings and results of the degradation study will need to be monitored and incorporated into actual design of future levee modifications as needed. Any risks and costs associated with degradation study recommendations, including any related to bridge scour, will be evaluated and addressed as part of that separate study effort.

- Impact of raised water surface profiles on Kansas River bridges. A large number of bridges cross the Kansas River within the current study area. Raising the system on either side of the river increases the future water surfaces, leading to higher lateral loads on these bridges, more potential for debris impacts, etc. A qualitative assessment was conducted using all known and available bridge inspection reports and information to identify the most highly impacted bridges and rank their potential for failure under multiple scenarios. Of critical concern to this study was the possibility that a bridge failure might directly cause a failure of the flood risk management system leading to inundation of the study area. The results of this assessment determined that if any of the affected bridges were to fail during a flood, there would be no direct failure impact to the adjacent flood risk system. Furthermore, the probability of a flood high enough to impact the bridges is small, and, even at that flood level, the probability of an actual bridge failure is small, making the overall scenario a remote and rare occurrence that does not justify the formulation of specific measures or alternatives. Additional detail of the bridge assessment is provided in Appendix A.

- Condition of existing CID floodwall foundations. At the beginning of the study there was significant uncertainty regarding the existing condition of the original timber pile foundations supporting the floodwalls in the CID unit. It was assumed they would be inadequate to support a raise of the walls. The walls would therefore need a large number of new adjacent piles to support buttressing of a raise, or the walls and foundations would need complete replacement. After several iterations of plan formulation and cost screening with this constraint in place, the team decided there was value in conducting excavation and testing of the foundation piles to address this uncertainty. Two locations were excavated and the piles were visually inspected and samples were laboratory tested. The resulting data reduced the uncertainty concern, and the subsequent structural analysis concluded that the existing piles were still capable of providing support of a raise, eliminating the need to consider wall replacement, and reducing the number of additional piles needed.

4.3 Management Measures

The following management measures were identified and studied for the applicability to each feature of the exiting unit and their ability to meet the project objectives.

4.3.1 No Action

In accordance with current policy it is necessary to fully evaluate the No Federal Action alternative for purposes of comparison to other alternative and future with-project conditions. Evaluation of the No Action plan is closely related to the future without project scenario and requires the projection of what course of action local entities may take given the lack of Federal involvement. It is possible that some of the recommended measures may be undertaken by the local sponsors. These local initiatives would likely to be focused on the engineering reliability measures which are the least costly of the recommendations offered herein, such as underseepage improvements. However, full implementation of the measures as described may not be possible with local budgets alone. The major requirements associated with structural feature reinforcements, and increased overtopping protection, are just as likely not to be accomplished under a local initiative. This would mean a significant long-term remaining risk.

The No Federal Action alternative does not address any of the project's objectives and does nothing to alleviate risks to public health and safety. While some local emergency preparedness plans can be updated and general awareness of the risks can be increased, this could be considered an inappropriate small scale response to significant life and safety risks.

Economic Impacts of the No Action Alternative. The economic implications of the No Federal Action alternative are broadly negative. The investment at risk within each unit is so large that No Federal Action will subject the study area to the possibility of an overall long-term adverse impact on the local economy, and dislocations of industry may even result. In the short term, with an absence of flooding, the current trends in-place for the local economy, tax base, population, and employment may remain intact. However, if major flooding occurred and one or more of the levee units failed, the long term effects are likely to include: diminished economic stability, business interruptions that could jeopardize workers jobs and wages, potential losses in population and employment, and reductions in the tax base (given net movement out the protected areas) and generally diminished property values.

The No Federal Action alternative would leave several of the busiest rail yards in the nation at significant risk. Levee failure(s) would halt or at least significantly impede the nationwide movement of goods by rail, and major interstate highways could also shut down. During any such failure, it is also expected that production centers, wholesale distribution, and containerized shipping centers would close. Following the flood, subsequent restoration periods could be months or years depending on the damage involved.

The No Federal Action alternative also raises the possibility of permanent loss of local manufacturing employment through industrial relocation to developing countries. Certain industries may see moving outside the United States as a more viable option in lieu of industrial re-investment and rebuilding after any widespread flood damage. Were this to occur, it could severely degrade the industrial base of the metropolitan area for decades.

Environmental Impacts of the No Action Alternative. The No Federal Action alternative results in no changes to the existing environment in and around the levee units unless catastrophic levee failure occurs. Levee failure could result in direct and indirect impacts through inundation of habitat of terrestrial populations. Direct impacts during flood events would be the displacement of mobile organisms and the loss of organisms unable to escape inundated areas. Indirect impacts would be the temporary or permanent loss of the already limited existing habitat preventing organisms from returning to the area.

Direct and indirect impacts could also result from the introduction of contaminants currently controlled or contained by businesses and industries in interior of the levee systems. While a complete inventory of chemicals and chemical classes is not available, the primary sources of contaminants within the Armourdale and CID levee units include auto salvage, railroad operations, electrical power generation, chemical plating, producers of starch and other household chemicals, a wastewater treatment facility, and additional sources of contaminants. Catastrophic levee failure and flooding within these units could result in the release of volatile organic compounds such as benzene, ethylbenzene, toluene, xylene, TCE (trichloroethene), and PCE (tetrachloroethene). Organic compounds released would minimally include fuels, grease, oil, plastic, and rubber. Inorganic compounds released would include metals such as chromium, copper, iron, lead, nickel, and zinc.

Contaminants in water can be transported within the water, transported into the atmosphere, absorbed into the soil/sediment or solid matter within the water, dissolved, degraded, and/or transformed. The release of contaminants from behind the levees due to catastrophic failure and flooding would cause significant immediate and long-term surface water, ground water, soil, and air contamination and result in carcinogenic and non-carcinogenic toxic effects to the human and natural environment. Flood response and recovery efforts would be hindered by the presence of released chemicals. The inhalation, ingestion, and contact with these materials would irritate the eyes, nose and throat. Prolonged exposure would lead to nausea, vomiting, headaches, dizziness, drowsiness and confusion. Relatively high concentrations would lead to respiratory failure, cardiac arrest, unconsciousness and death.

Impacts from the No Federal Action alternative could range from no significant impact under non-flood events, to minor or significant impact during flood events, depending on the location of levee failure and the resulting duration of inundation.

4.3.2 Non-Structural Measures

Floodfighting. This measure attempts to address all objectives through temporary means implemented during a flood event aimed at preventing or minimizing flood damages.

Relocation or flood-proofing of individual structures. This measure aims to reduce or prevent damages in the study area by removing structures or preventing floodwaters from entering them. It does not address any of the objectives specific to the existing system (i.e. overtopping or structural and geotechnical reliability of the existing features).

4.3.3 Structural Measures

Tree clearing and/or channel modification. The related objective addressed is inadequate reliability against overtopping. Channel modification would be aimed at attempting to establish a more efficient cross-sectional flow area along substantial lengths of the levee foreshore to allow a greater discharge capacity, and lower the water surface profile of the design flood.

Modify or replace existing pump stations. The related objective addressed is inadequate reliability against structural failure. All pump stations will be initially evaluated using current criteria and required factors of safety for uplift, strength, and hydraulic capacity. Those found not meeting criteria for any of these failure modes will be proposed for modification or replacement. If evaluation shows that the original purpose of the pump station is no longer required for operation of the project, the pump station will be recommended for abandonment.

Modify or replace existing floodwalls. The related objective addressed is inadequate reliability against structural failure. All existing floodwalls will be initially evaluated for strength, stability, overturning, and foundation reliability. Any floodwall not meeting criteria for any of these failure modes will be proposed for modification or replacement.

Replace or expand underseepage control features. The related objective addressed is inadequate reliability against geotechnical underseepage failure. Each unit will be initially evaluated using current criteria and required factors of safety for underseepage. Areas showing low reliability for this failure mode will be proposed for replacement or expansion of existing underseepage control features, or if no existing features are present, new installations. Underseepage control is typically achieved through the use of area fill, impervious berms, underground slurry cut-off walls, buried collectors, or relief wells.

Unit Raise. The related objective addressed is inadequate reliability against failure due to unit overtopping.

- Raises of earthen levees typically maintain the existing side-slope profile, resulting in a widening of the levee footprint, often to one side or the other (landside or riverside), or possibly in both directions. If levee width increases are not possible, other methods available include adding retaining walls to limit width increase, adding floodwalls on top of the levee, or replacing the levee with a floodwall.
- Concrete floodwall raises are typically achieved through structural modification, as long as the existing wall base and foundation can provide sufficient support. If modification is not possible, the wall can be removed and replaced with a higher wall on a new foundation.

New CID floodwall tieback. This measure was added for consideration in the CID Unit after the first iteration of screening and alternative formulation. The related objective addressed is to economically achieve reliability against all potential failure modes. The floodwall at the upstream end of the CID Unit has been raised previously and would require very expensive modifications for additional raise, or possibly a complete replacement. A higher floodwall also requires a large number of new underseepage relief wells. The proposed measure consists of constructing a new wall to tie the existing unit into the bluff at a different location, thus eliminating the cost of modification or replacement of a long reach of the existing unit. This would result in a portion of the study area where the current flooding risk would remain; however, this area contains only railroad tracks and no businesses or residences, ensuring no continued life safety risk.

4.3.4 Conclusions from Initial Screening of Measures

4.3.4.1 *Floodfighting*

Flood fighting is generally best thought of as an aid to manage unpredictable and unforeseen problems during flood events. For large levee units where substantial investment is protected, some flood fighting can be planned and implemented for limited low-risk situations. But, in general, when exposed to massive flood events, flood fighting measures will often prove unreliable. For the levee units and problems under examination in this study, flood fighting is generally not an acceptable planning alternative when compared to engineered solutions. Flood fighting generally will not prevent underseepage failures when dealing with very high pressures, nor can flood fighting reliably prevent structural floodwall failures under extreme load conditions. Nor is flood fighting a reliable option for substantially raising the elevation of a large levee unit.

4.3.4.2 ***Non-Structural Measures***

Nonstructural approaches have merit when the site characteristics and the flooding threat are compatible with the nonstructural capabilities. The intent of non-structural measures is not to prevent the flooding from occurring, but to reduce the damages and consequences caused by the flooding. Typical methods include structure removal or relocation, structure elevation, or flood-proofing, either wet or dry. Wet flood proofing allows water to enter the structure but focuses on reducing the damages caused, while dry flood proofing aims at keeping floodwaters outside the structure.

Structure Removal or Relocation. Within the Armourdale and CID Units there are 951 residential structures and 1,043 businesses and public facilities with a total value of approximately $4.27B (October 2012 prices). Due to the large number, density, and value of homes and businesses within these levee units, structure acquisition for removal or relocations is not efficient or acceptable.

Structure Elevation. Structure elevation may provide some protection from moderate floodwaters, but would be inefficient at preventing significant flood damages such as those associated with catastrophic levee failure. The cost of elevating existing buildings is higher than the cost associated with implementing higher building standards for new construction. The estimated cost to elevate an existing home (FEMA 2009), in 2009 dollars, ranges from $30 to $100 per square foot, depending on the type of home and the amount of raise, up to eight feet. Assuming an average home size of 1,000 square feet results in a preliminary cost range of $29M to $95M. The cost to elevate commercial or industrial buildings (if feasible) would be higher.

Structure Flood Proofing. Flood proofing measures generally have limited application where a large number of homes and businesses are located within the flood prone area; and flood proofing of areas below the 100-year flood or base flood elevation (BFE) in residential buildings is not permitted under the National Flood Insurance Program (NFIP), except in communities that have been granted an exception to permit flood-proofed basements. NFIP allows new or substantially improved non-residential buildings in the 100-year floodplain to have a lowest floor below the BFE, provided the design and methods of construction have been certified by a registered professional engineer or architect as being dry flood proofed in accordance with established criteria.

The costs associated with flood proofing existing buildings are also higher than the cost associated with implementing higher building standards for new construction, and the feasibility of flood proofing existing buildings varies based on site and structure constraints. The estimated cost for wet flood proofing an existing home (FEMA 2009) in 2009 dollars, can range from $2 to $17 per square foot depending on the type of structure and the height of flood proofing effort, up to eight feet. Assuming an average home size of 1000 sq ft. results in a preliminary estimate of $1.9M to $16M for residential structures only. Dry flood proofing costs for commercial structures can vary widely depending on structure type, and is generally considered to only be effective up to three feet. The expected flood depths in the Armourdale and CID units would significantly exceed three feet. Due to the large number of older existing homes and businesses within these levee units and the significant depth of flooding expected by levee failure, flood proofing is not considered efficient or effective.

Other Measures. There are other types of non-structural measures focused on informing and warning the public, removing the public from harm's way, and preventing further development within the area of risk. These include flood-warning systems, floodplain management planning, including emergency action and evacuation plans, and municipal ordinances prohibiting or limiting development. The Kansas and Missouri Rivers are heavily monitored by both the Corps of Engineers and the United States Geological Survey and forecasts of expected river conditions are regularly issued by the National Weather Service. These activities minimize the effectiveness of typical flood-warning systems using gauges and sirens. The local non-Federal sponsors and municipalities in the project area have existing floodplain, emergency, and evacuation plans of varying levels of detail. The multiple sponsors with the assistance of the Kansas City District, are currently engaged in an effort to coordinate these various plans and identify those areas were additional detail and plan development may be necessary. The Corps of Engineers will continue to provide public information, technical, and financial assistance to these efforts beyond this Feasibility Study, as allowed by current authorities and programs. Floodplain ordinances and building codes are recommended, however their implementation is the responsibility of the local municipal governments.

Non-Structural Conclusion. There is already an extensive existing structural flood risk management system providing benefits to the study area. The nature of damages expected from levee failure under the existing condition, and the need and desire for large-scale future risk reduction within the study area, especially from system overtopping, far exceeds the normal performance parameters of typical nonstructural measures. The value of the dense urban development in the study area precludes consideration of large scale relocation, elevation, or flood-proofing of structures. For these reasons, it was concluded that without structural modification of the existing levee system these methods alone would not provide the desired performance improvements and they were not carried forward for further analysis. It is recognized that there may be possibilities and uses for nonstructural measures in addition to, and coordination with, structural alternatives, especially in limited locations for the prevention of damages due to localized interior flooding or for the protection of infrastructure of local importance. These potential limited applications would be best identified and pursued independently by the project sponsors.

4.3.4.3 *Non-Raise Structural Alternatives*

The management measures discussed previously would be combined to improve the levee system reliability by implementing modifications to structural features (pump stations, gatewells, closures, floodwalls, etc.) and improvements to the underseepage control system, without raising the existing height of the levee units. The different available methods for structural modification and underseepage control allow for the development of multiple alternative plans under a no-raise scenario.

Channel modification was also evaluated as a separate non-raise alternative plan. Channel modification was modeled for both sides of the Kansas River through the study area and the results indicated some additional conveyance capacity under modified conditions. However, the conveyance gains are very limited (not totally effective and complete) and do not fully serve to establish the desired design discharge.

Furthermore, it is expected that channel modification would have a limited life much less than the 50-year period of analysis. The natural process of meandering and foreshore building would require repeated dredging cycles to maintain the expanded floodway. The overall prospect of massive environmental disruption, extensive maintenance dredging adjacent to the existing levees, the potential creation of new underseepage paths, and the general risk associated with effective timing of dredge cycles and potential floods make the channel-modification measure undesirable.

4.3.4.4 ***Unit Raise Structural Alternatives***

This group of alternatives would improve levee system reliability by implementing the modifications to structures and underseepage control necessary to address identified weaknesses in combination with raising the height of both units. These plans address all potential failure modes. The different available methods for structural modification and underseepage control allow for development of multiple alternative plans for screening under a unit raise scenario. As the unit height is increased, there are many dependencies and conflicts created among the various types of management measures identified. The alternative plans under this scenario must consider all of the following concerns:

- As the levee height is increased, stress on the adjacent structural and geotechnical features also increases, causing associated changes in the scope and viability of the different management measures.
- Measures considered for one feature may cause impacts, either positive or negative, to other features.
- Some existing features which can be modified with no raise, or even a short raise, may need replacement at a higher raise. Similarly, different underseepage control methods will perform differently, and may lose effectiveness, when a raise is considered.
- The raise of an earthen levee requires an expansion of the levee footprint and the need for additional permanent right-of-way on one or both sides of the levee. Considering the urban development of the study area, this is not possible in all levee reaches. Levee raises may need to be constructed with retaining walls to limit footprint expansion, installation of floodwalls on top of the levee, or completely replacing the levee with a new floodwall.

4.4 Plan Formulation and Evaluation

The Interim Feasibility Report recommended improvements to four units of the system: the Fairfax-Jersey Creek, North Kansas City, and East Bottoms Units on the Missouri River, and the Argentine Unit on the Kansas River. The Missouri River Units were determined to have adequate height to resist overtopping at the design flood level, but require significant underseepage and structural modifications to maintain acceptable overall system reliability. In addition to similar geotechnical and structural reliability concerns, the entire Kansas River portion of the system was determined to be of insufficient height to provide adequate overtopping protection. The Interim Report included the detailed analysis of alternatives for the Argentine Unit. The Argentine NED plan was identified as a unit raise that provided a 64% reliability to pass the 0.2% event. This level of flood risk management is consistent with the Missouri River units, and meets economic project justification criteria.

The Armourdale and CID Units are located immediately downstream of the Argentine Unit. In accordance with the planning objective of desiring to achieve a uniform system level of flood risk management and to reduce the potential for induced damages between units within the system, the evaluation of raise alternatives sought a plan that provided at least the same reliability as the Argentine Unit project at the 0.2%-chance water surface profile.

4.4.1 Non-Raise Structural Alternative Plans

While a non-raise structural alternative plan would provide improvements to the structural and geotechnical reliability of the units at their current height, they would not achieve a uniform system level of flood risk management benefit, and reduce the potential for induced damages between units within the system. A no-raise plan would not be consistent with the authorized plan for the upstream unit in the system, or the desired conveyance target.

4.4.2 Unit-Raise Structural Alternatives

Based on the stated study objectives and evaluation criteria, the highest priority was placed on evaluation and screening of the unit-raise structural alternatives. To provide the desired reliability at the 0.2% event results in an actual physical raise above the existing height of between 1.2 to 5.2 feet in the Armourdale Unit, and 0.2 to 3.9 feet in the CID Unit.

The management measures for structural and geotechnical components were evaluated for their feasibility and effectiveness under the future expected hydraulic conditions. The alternative plans combining the most effective measures were retained for further cost-estimating and economic analysis. By focusing first on the required raise, the study team was able to quickly evaluate whether the desired levee height was technically feasible before evaluating lower elevation alternatives. Discussion and results of these evaluations specific to each unit are presented in the following sections.

4.4.2.1 *Central Industrial District Unit*

Floodwalls. Investigation and engineering analysis confirmed that the floodwall sections can be modified to support the additional increase in height without need for replacement.

Levees. Sufficient real estate is available in the levee reaches for the expansion of width associated with an earthen levee raise.

Underseepage. To control underseepage at the new levee height, area fill to raise the landside ground elevation is proposed for reaches with sufficient real estate availability. In more congested reaches, both relief wells and slurry cut-off walls were feasible at the selected raise. A preliminary estimate of the relative cost of these two measures was calculated for comparison. The overall life-cycle cost of the relief wells was found to be less than the construction of a slurry cutoff wall. The slurry cut-off wall measure was eliminated and only relief wells were included in the final alternative plans.

Existing Pump Stations. The existing pump stations were evaluated for their ability to withstand increased hydraulic uplift pressures and handle flows from additional relief wells. Several stations were found to need strength and capacity modifications, and two smaller stations were determined to be no longer needed and are proposed for abandonment. Since these modifications are primarily driven by the new levee height, they are necessary in any final alternative plan.

Unit Tieback. The inclusion of a new tie-back connection between the existing unit and the river bluff was determined feasible on several different alignments. Where the tieback connection is located along the existing alignment impacts the resulting number of new relief wells needed, whether or not a new pump station is required to handle the relief well flows, and the number and locations of new railroad crossings and closure structures. These options allow for several alternative plans to be considered for the final analysis.

4.4.2.2 *Armourdale Unit*

Due to the varying existing conditions and characteristics of the study area, it was necessary to separately evaluate alternative plans for discreet reaches of the overall unit. For underseepage evaluation, the unit was divided into reaches of similar geotechnical conditions (unit height, impervious soil blanket thickness, aquifer thickness, seepage entrance condition, etc.). Evaluation of the different raise measures in each reach considered existing protection (levee vs. floodwall), adjacent development, the potential for real estate conflicts, and potential encroachment into known areas of Hazardous, Toxic, and Radioactive Waste (HTRW) contamination.

Floodwalls. Each reach of existing floodwall was evaluated to determine if the existing wall and foundation were adequate, or could be modified successfully, to support additional height. Analysis indicated that the existing floodwall from stations 60+40 to 70+80 cannot be modified to support a raise leaving only replacement as a viable measure. The existing floodwall from stations 257+65 to 302+58 can be modified to support the raise, except for the section from 274+36 to 277+21 that must be realigned to avoid conflicts between new closure structures and the Missouri Pacific and Union Pacific Railroad Bridges. A new section of raised floodwall incorporating new closures to replace the existing wall is the most feasible alternative at this location.

Levees. Where possible, earthen levee raises are the preferred, and typically least cost, alternative. As discussed previously, earthen levee raises create a wider levee footprint, either landside or riverside of the existing levee. In almost all reaches of this unit, the levee is immediately adjacent, or integral, to the Kansas River bank slope, eliminating the possibility of a riverside raise. Landside levee width increases are significantly complicated by the potential for real estate conflicts with adjacent businesses, railroads, utilities, pump stations, and areas of environmental concerns. Additionally, in some reaches, certain measures would create limitations to unit access needed for maintenance, inspection, and operation.

In the initial evaluation of alternatives in each reach, the PDT decided to eliminate all levee raise alternatives that caused encroachment on adjacent buildings, infrastructure, and known areas of environmental contamination. In the reaches where earthen levee raise would not fit this constraint, the evaluation next considered levee raises with retaining walls to limit width increase, then floodwalls on top of the existing levee, and finally replacing the levee entirely with a new floodwall. The evaluation of alternatives thus focused on the avoidance of real estate relocation costs. If the conflicts were strictly concerns of real estate easements or project access, the alternatives were retained for the final evaluation.

Those levee reaches with no HTRW or real estate concerns were only evaluated to be raised by typical earthen levee methods. All other alternatives in these reaches were eliminated from further consideration.

The existing unit ends at Station 61+00LE. Downstream of Station 42+00LE the required levee height increase to match the desired reliability is less than 1.2 feet. While a landside levee raise is technically feasible in this reach, the implementation of this raise will conflict with the adjacent active railroad track. This single track is the only existing infrastructure receiving benefits in this reach. On the opposite side of the track is the high ground embankment of an existing city street (James Street). It is feasible to tie into this existing high-ground with a sand bag closure structure across the railroad track at Station 42+00, this shortening the unit by almost 2,000 linear feet. The existing levee would remain in place providing benefits to the lower reach up to its current elevation. If this reach is overtopped, the sand bag gap would prevent floodwaters from backing up into the rest of the study area. This alternative would eliminate the railroad impacts of a raise. Both alternatives for this reach are retained for the final evaluation.

Underseepage. Three reaches of the unit were identified as potentially not meeting required underseepage factors of safety under the proposed future raise. In all identified reaches, the proximity of urban development eliminated consideration of additional or expanded berms. The evaluation next considered the use of pressure relief wells or a slurry cut-off wall at each location. However, similar to the CID-KS evaluation discussed previously, the cost of a slurry wall installation was found to be greater than the life-cycle cost of relief wells, making the use of wells the preferred alternative. In the reach from Station 295+00 to 313+00, thirty-five new relief wells are needed to address increased hydraulic pressures. In the reach from station 62+00 to 82+00 adjacent railroad tracks and facilities would need to be relocated for relief well installation. Additionally, this reach overlaps with an identified groundwater contamination concern between stations 45+00 and 75+00. A slurry cut-off wall constructed to bedrock is the only remaining option. In the reach from 257+65 to 295+50,which includes the railroad "slot", the existing ground surface is significantly lower than surrounding areas. Placement of area fill in this low area was deemed sufficient to improve underseepage safety.

Pump Stations. Six pump stations require modifications due to either insufficient strength, potential for flotation, or inadequate capacity to handle relief well flows. Two additional pump stations are no longer needed as the facilities they were built to service are no longer in existence. These pump station modifications are necessary regardless of how the unit is raised and are common to all final alternative plans.

4.4.3 Initial Economic Analysis

Preliminary economic analyses were prepared in 2006 to assist in the screening of the initial array of alternatives. As stated previously, the selection of management measures and development of alternatives was limited by the Planning Objective of achieving and maintaining a uniform level of flood risk management for the Kansas Citys system. The Argentine NED plan recommended in the Interim Feasibility Report was applied as the desired target for system performance in the Kansas River units. Costs and benefit estimates were prepared for two other scales of levee raises and associated modifications leading up to this target. Although these lower raise alternative plans do not meet all of the study objectives they were necessary for comparison to ensure the identification of the plan, or plans, that meet economic criteria within each levee unit and the overall system. The economic analysis of the three Kansas River raise alternatives evaluated are shown in Table 4-1 with Kansas River Plan 3 (KR3) representing the plan consistent with the Argentine Unit NED plan. Cost estimates for the Argentine Unit were included to allow for comparison of the Kansas River three-unit system total.

Table 4-1: Screening Analysis of Alternative Raise Profiles

Unit	First Cost	Total Annual Cost	Total Annual Benefits	B/C	Net Benefits
Kansas River Plan 1 (KR1)					
Argentine	$33,042,548	$2,093,795	$16,322,473	7.80	$14,228,678
Armourdale	$51,723,299	$3,371,286	$5,234,014	1.55	$1,862,728
CID-KS	$39,959,191	$2,563,797	$3,266,651	1.27	$702,854
TOTAL	$124,725,038	$8,028,878	$24,823,138	3.09	$16,794,260
Kansas River Plan 2 (KR2)					
Unit	**First Cost**	**Total Annual Cost**	**Total Annual Benefits**	**B/C**	**Net Benefits**
Argentiue	$33,945,404	$2,150,335	$16,560,871	7.70	$14,410,536
Armourdale	$61,233,118	$3,984,373	$5,553,332	1.39	$1,568,959
CID-KS	$40,482,623	$2,597,032	$3,454,202	1.33	$857,170
TOTAL	$135,661,145	$8,731,740	$25,568,405	2.93	$16,836,665
Kansas River Plan 3 (KR3)					
Unit	**First Cost**	**Total Annual Cost**	**Total Annual Benefits**	**B/C**	**Net Benefits**
Argentine	$35,313,745	$2,278,318	$17,081,997	7.50	$14,803,679
Armourdale	$63,411,583	$4,138,267	$5,744,664	1.39	$1,606,397
CID-KS	$41,759,697	$2,686,581	$3,608,586	1.34	$922,005
TOTAL	$140,485,025	$9,103,166	$26,435,247	2.90	$17,332,081

Note: Oct 2005 prices; 5.125% interest rate (Prepared Feb 2006)

The 2006 screening results indicated that the total net benefits of the three-unit Kansas River system were continuing to rise at the KR3 plan. Traditional economic analysis requires the identification of the plan that maximizes the net economic benefits, defined as the National Economic Development (NED) Plan, which usually means analyzing progressively larger plans until it is shown that net benefits have begun to decrease. The screening results indicated that the NED plan for these units would be somewhere above the plan identified as the preferred maximum, which also meant that it would be inconsistent with uniform system performance. In consultation with the project sponsors it was agreed that it was not desired to continue the analysis of larger plans to identify the NED, as allowed by the Categorical Exemption to NED Plan stipulated by ER 1105-2-100, Section 3-3.b.(11), which states:

> *For flood damage reduction studies, where the non-Federal sponsor has identified a desired maximum level of protection, where the with-project residual risk is not unreasonably high, and where the plan desired by the sponsor has greater net benefits than smaller scale plans, it is not required to analyze project plans providing higher levels of protection than the plan desired by the sponsor.*

The results of the initial economic analysis and application of the Categorical Exemption supported the selection of the KR3 plan as the system levee height for alternative formulation. Further alternative evaluations and comparisons for the Armourdale and CID units focused on the development and refinement of plans to implement this selected raise plan and address the associated underseepage and stability concerns.

4.4.4 Efficient Combinations of Measures and Scales

4.4.4.1 *Central Industrial District Unit Alternative Plan Development*

Six alternative plans were retained for the final evaluation. Each plan includes the same raises of the earthen levee and floodwall sections, the same area fill locations, and the same pump station modification and abandonments. The differences among the plans are related to the new tieback measure; whether or not this measure is included, where the tieback connection is located along the existing alignment, the effect of the new tieback on the proposed relief well system, and what alignment the tieback is constructed on between the existing unit and the bluff. The six alternatives are described briefly as follows:

#1 Unit stops at Sta 130+00 and turns to bluff (adds 4 stop log gaps and 15 new relief wells)
#2 Unit continues to Sta 166+80 (adds 83 relief wells/new pump plant/1 stop log gap)
#3 Unit stops at Sta 138+95 and turns to bluff (adds 2 stop log gaps and 30 new relief wells)
#4 Unit stops at Sta 130+00 and turns to bluff (adds 4 stop log gaps/smaller pump station)
#5 Unit continues to Sta 166+80 (adds 83 relief wells/new pump plant/1 stop log gap/header pipe)
#6 Unit stops at Sta 138+95 and turns to bluff (adds 2 stop log gaps and 30 new relief wells with a new pump plant)

The primary differences between the six plans in the final array is whether or not to modify and raise the existing floodwall upstream of station 130+00, or to essentially shorten the unit by constructing a new tieback to the bluff along the eastern edge of the study area. The existing floodwall in this reach has already been modified and raised in the past. Although the foundation analysis determined that additional raise could be supported, the actual implementation would be technically very complex. The area inside the unit along this reach contains multiple railroad tracks and one abandoned and dilapidated railroad storage warehouse which provide limited economic benefits.

Note that Alternative Three and Alternative Six are identical in terms of floodwall tieback location and height, number of closure structures, and number of relief wells. The thirty relief wells will be designed to surface discharge and thus will provide the same degree of underseepage control in the future-with-project-condition, with or without a pump station. At the time the alternatives were first developed and screened it was unknown if a pump station would be needed in order to prevent interior flooding damages as a result of the relief well flows. The pump station analysis is summarized in Section 4.4.4.3.

Similarly, Alternative Two and Alternative Five are identical with the exception of the header pipe included in Alternative Five. Alternative One and Alternative Four each stop at Station 130+00 and turn to the bluff and each include four additional stop log gaps. Alternative One also includes 15 new relief wells, whereas Alternative Four includes a smaller pump station but no new relief wells.

No Tieback: Alternatives Two and Five assume that the existing wall is raised and no tieback is constructed. Each plan includes a new pump station to handle the flow from the additional 83 relief wells and a new stop log closure structure constructed upstream of the existing closure at the end of the unit. Alternative Five has a different configuration of header piping to collect flows from the relief wells. Both alternatives have the same future with and without project conditions. Implementation of either alternative will provide reliable flood risk management up to the recommended top of levee elevation along the full extent of the existing unit alignment. Without project implementation, the reliability of the unit does not meet current criteria and the entire CID study area is subject to inundation from flood events less than the system design event. These alternatives meet all project objectives and are within the project constraints.

Tieback at Sta. 130+00: Alternatives One and Four assume that a tieback is constructed to the bluff starting at Sta. 130+00, immediately downstream of the Kansas City Terminal Bridge. The existing floodwall upstream of Sta. 130+00, including the existing stop log closure at the KC Terminal Bridge, would not be raised. The tieback would require four new stop log closure structures and 15 new relief wells. Alternative Four assumes that a new small pump station would be needed to handle additional relief well flows. Alternative One does not include a pump station. Both alternatives have the same future with and without project conditions. Implementation of either alternative will provide reliable flood risk management up to the recommended plan top of levee elevation along the existing unit alignment downstream of Station 130+00. Upstream of this location, the existing floodwall would remain in place and continue to provide benefits up to its current elevation. If a flood exceeded this height, this reach would overtop causing inundation of the railroad tracks. The new tieback would prevent these floodwaters from entering the rest of the study area.

Tieback at Sta. 138+95: Alternatives Three and Six assume that a tieback is constructed to the bluff starting at Sta. 138+95. The existing floodwall upstream of this location would not be raised. Under both alternatives, the existing stop log closure at the Kansas City Terminal Bridge would be raised. The tieback itself would be shorter than in other alternatives, and require only two new stop log closure structures. However, an additional 30 new relief wells are needed. Alternative Six assumes that a new pump station would be needed to handle additional relief well flows. Alternative Three does not include a pump station. Both alternatives have the same future with and without project conditions. Implementation of either alternative will provide reliable flood risk management up to the recommended plan top of levee elevation along the existing unit alignment downstream of Station 138+95. Upstream of this location, the existing floodwall would remain in place and continue to provide protection up to its current elevation. If a flood exceeded this height, this reach would overtop causing inundation of the railroad tracks. The new tieback would prevent these floodwaters from entering the rest of the study area. Without project implementation, the reliability of the unit does not meet current criteria and the entire CID study area is subject to inundation from flood events less than the system design event. These alternatives meet all project objectives and are within the project constraints.

4.4.4.2 *CID Cost Screening Evaluation*

In July 2008, screening level cost estimates were prepared for the six final alternatives. The results are presented in the following Table 4-2.

Table 4-2: CID-KS Screening Cost Estimates

Alternative Plan	Preliminary Cost ($M)
#1	$ 98,624
#2	$ 130,026
#3	$ 89,918
#4	$ 102,580
#5	$ 130,834
#6	$ 96,136

Note: October 2008 Prices

4.4.4.3 *CID New Pump Station Analysis*

Following the initial plan evaluation and cost estimates, further analysis was conducted to determine the technical necessity of a new pump station to handle relief well flows. A review of the existing interior storm drainage system showed that if all proposed new relief wells were installed as surface discharging, there was adequate capacity to carry the expected flows to existing sewer outlets and pumping facilities. Removing the new pump station from the proposed alternative plans eliminates Alternatives Four, Five, and Six from consideration (they are now identical to Alternatives One, Two, and Three, respectively). Furthermore, with no pump station the estimated cost of Alternative Two is reduced by approximately $8.9 million, for a new estimate of $121.1 million. The estimates for Alternatives One and Three are not affected by this pump station evaluation.

4.4.4.4 ***CID Floodwall Foundation Investigation***

In December of 2010, an investigation was conducted of the existing condition of the timber pile floodwall foundations. The intent of this investigation was to address one of the Key Uncertainties previously identified. In each reach of existing floodwall, an excavation was made on the landside of the wall to expose the existing timber foundation for inspection and analysis. The excavations were at approximately Stations 30+00 and 114+00. At each location the piles were inspected and their condition documented. Sonic Echo Methods/Impulse Response (SE/IR) was used to estimate the length and soundness of the piles. Ultrasonic Pulse Velocity (UPV) testing was attempted at one location with no useful results, and Sonic Pulse Velocity (SPV) testing was used at both locations. Wood cores were obtained from Stations 114+00 for laboratory testing to determine specific gravity, moisture, creosote penetration, and fungal testing.

The results of the inspection and analysis indicated that the existing foundation was in good condition. This led to a revised assumption by the study team concerning the ability of the existing foundation to provide support for floodwall modifications. The previous assumption had been that the foundation would not be able to provide support for modifications and that the existing walls would need complete replacement.

The change from floodwall replacement to modification decreased the cost of all three remaining alternative, but did not alter their relative ranking. Alternative Three is still the lowest cost alternative plan, there by maximizing the net economic benefits. Alternative Three was retained as the Recommended Plan for the Central Industrial District Unit.

4.4.4.5 ***Armourdale Unit Alternative Plan Development***

Alternative evaluation determined that, in most of the reaches of the unit, only one alternative plan was identified as technically feasible and effective to perform the desired raise and address the respective impacts to appurtenant structural and geotechnical features. These individual reach alternatives are thus common to all final alternative plans for the overall unit. Similarly, structural and hydraulic pump station modifications are necessary based on the new unit height and are common to the final array of plans.

The final evaluation of alternatives thus focused only on those unit reaches where more than one feasible alternative was identified and carried forward. In five separate reaches of the unit, multiple raise alternatives were identified as feasible. These reaches and their alternatives are shown in Table 4-3.

Table 4-3: Armourdale Reaches for Further Evaluation

Start Station	End Station	Remaining Alternatives
10+00UE	16+48UE	1. Landside levee raise. 2. Riverside levee raise. **3. Replace levee with floodwall.**
77+80	81+00	1. Landside levee raise 2. T-wall on levee. 3. **Replace levee with floodwall.**
95+00	105+00	1. Landside levee raise. **2. T-Wall on levee**
120+00		1. Landside levee raise **2. Landside levee rise with retaining wall**
230+00		1. Landside levee raise **2. T-Wall on levee**
240+00	257+66	1. Landside levee raise. 2. T-Wall on Levee. **3. Replace levee sections with floodwall.**
42+50LE	61+00LE	1. Landside levee raise. **2. New sandbag gap closure at Sta. 42+50LE.**

Note: **Bold font** indicates selected alternative.

In the majority of these remaining reaches, the remaining technically feasible alternatives create access limitations and real estate related conflicts that could require potentially costly relocations. Experience on similar projects in the Kansas City area, and other locations, has shown that real estate access and relocations involving railroads are both very costly and time consuming. This is an important consideration in the final alternative evaluation and selection. Following is a brief discussion of the alternatives in each reach.

Sta. 10+00UE to 16+48UE. A landside levee raise would require relocation of railroad tracks and a riverside levee raise would require modification of two large outfall structures. Replacement of the existing levee with a floodwall eliminates all real estate conflicts. Alternative Three is recommended.

Sta. 77+80 to 81+00. A landside levee raise would require relocation of railroad tracks. A T-wall on the levee limits top of levee road accessibility to this area of the unit. The access cannot be rerouted to the landside due to the railroad tracks. Replacement of the levee with a new floodwall eliminates the real estate conflicts and maintains access. Alternative Three is recommended.

Sta. 95+00 to 105+00. A landside levee raise would encroach upon an area needed for access to an adjacent business, Kansas City Hardwoods. A T-Wall on top of the levee limits top of levee road access, but access could be rerouted on the landside in the same area as the business access. Alternative Two is recommended.

Station 120+00. A landside levee raise would encroach upon an adjacent business, KC Railcar. The use of a retaining wall at the landside two would limit the increase in levee width and avoid this conflict. Alternative Two is recommended.

Station 230+00. A landside levee raise would encroach upon an adjacent business, Sambol Meat Packing. A T-wall on top of the levee would eliminate the increase in levee width and avoid this conflict. Alternative Two is recommended.

<u>Sta. 240+00 to 257+66.</u> This reach contains two existing levee sections separated by an existing floodwall section. The floodwall has already been identified for replacement as its foundation cannot support modification for a raise. A landside raise of the levee sections would encroach upon areas used by adjacent businesses for storage and access. A T-Wall on top of the levee would limit top of levee access. Landside access in this reach is already difficult due to the operations of multiple adjacent businesses and the Kansas Ave. bridge approach. Replacement of the levee sections with new floodwall eliminates the real estate conflicts, creates additional area for landside access, and provides for a uniform raise measure for the entire reach. Alternative Three is recommended.

<u>Sta. 42+50LE to 61+00LE.</u> Even though a landside levee raise would be a very short increase in height, access and implementation of the project would conflict with the adjacent railroad track. A new sandbag gap closure at Sta. 42+50 eliminates this minor unit modification and potentially costly real estate conflict. Alternative Two is recommended.

The evaluation of the technical alternatives in each discrete reach of the Armourdale Unit resulted in only one feasible method of achieving the levee height increase and address associated structural and geotechnical impacts. The combination of alternatives in each reach results in one complete alternative plan for the Armourdale Unit to meet the study objectives and constraints. Thus, there are no other plans for a cost screening evaluation. The remaining Alternative Plan is the Recommended Plan for the Armourdale Unit.

4.4.5 Formulation Criteria

Planning objectives and early economic analysis led to the determination of the KR3 levee raise plan as the desired level of flood risk management. All subsequent alternatives formulated provide the same level of future economic benefit to the study area. The evaluation and comparison of the final array of alternatives focused on those alternative measures and plans that maximized the cost effectiveness of the project, thereby increasing the net economic benefit. Economic screening and evaluation was conducted in 2008 and used the prices and interest rates current at the time.

Screening level cost estimates and estimated construction periods for each of the alternatives were developed in accordance standard Corps of Engineers estimating practice. Interest during construction (IDC) for each alternative was calculated based on the total first cost for each alternative, the starting and completion dates for each phase, assumed equal monthly expenditures during each phase, and the Federal interest rate of 5.375%. Potential Federal funding constraints were not considered in the starting and completion dates of the implementation phases; appropriate funding was assumed available for each phase.

The total first cost for each alternative includes the estimated construction cost, cost for lands, easements and rights of way, preliminary engineering and design cost, supervision and administration cost, and contingencies. Interest during construction calculated for each alternative was then added to the total first cost to derive the economic cost of each alternative. The economic cost was then annualized for a 50-year period of analysis and a 5.375% interest rate. Other direct costs of project implementation (such as potential induced damages) were determined and included in the total annual project implementation cost.

4.5 Results of Plan Formulation and Evaluation

4.5.1 Costs for Operation, Maintenance, Repair, Rehabilitation, and Replacement

Operation, Maintenance, Repair, Rehabilitation, and Replacement (OMRR&R) costs were estimated for each alternative and are based on a life cycle cost analysis. The analyses include only the additional OMRR&R costs that the sponsors would be expected to incur based on the proposed unit modifications. The analyses considered and accounted for the additional OMRR&R in each year of occurrence, and then computed a present worth value of the future OMRR&R costs. The present worth value was then annualized using a Federal Interest Rate of 3.5% and a 50 year period of analysis. Following are the major assumptions used in determining the additional OMRR&R costs that the local sponsors would incur with each alternative.

- New Relief Wells: Each new well is assumed to be maintained every four years at an estimated cost of $5,000 per well. New wells are assumed to be replaced after 40 years at a cost equal to the current construction cost; the replacement cost includes 10% E&D and 7% S&A. The sponsor would continue to incur costs for any existing relief wells but these costs are ongoing for the existing project and are not included in the analysis of the proposed project.

- The levee units in the Kansas Citys project are well-maintained and the sponsors comply with annual inspection requirements. It is assumed that the sponsor's current OMRR&R costs for the existing project will continue.

4.5.2 Other Economic Benefits Not Quantified

The Corps of Engineers benefit evaluation process involves analysis of the economic losses to the subject study area from flooding, and the potential gains to the study area from the successful prevention of flooding. Some of the economic impacts that are likely to occur in the "without project" condition may be of major significance to a metropolitan area or community, but may not have any net impact on the national economy. For example, if a flood interrupts production at a given business in one community, that community suffers a loss. However, if the lost production is replaced by production at another plant elsewhere in the country, the loss to the local community does not represent a net loss to the national economy. These regional (RED) impacts are not included in determining the NED benefits and costs, but should receive consideration in the overall decision-making process.

In the Kansas Citys study area, some major production facilities are either a sole producer of a specific product or are one of just a very few in the nation that produces that product. Proctor and Gamble is a prime example in the Armourdale Unit. Loss of production capability in these instances could be an economic loss to the nation unless consumers were able to find a similar product and made the choice to purchase the substitute product. However, these potential NED losses were not quantified for purposes of this study.

4.6 Plan Selection

4.6.1 Verification of the Systems Analysis

As previously presented in Table 4-1, a comparison of costs and benefits of different system raise alternatives was conducted in 2006 and identified the KR3 plan as the preferred raise maximizing the net economic benefit for the system within the Planning Objectives and Constraints and the desires of the Non-Federal sponsor.

During the economic analysis of the Recommended Plan it was recognized that the current annual costs and benefits are significantly higher than in the 2006 screening, especially in the Armourdale Unit, even after adjustment for inflation. The earlier calculation of economic benefits was derived from overtopping failure impacts only. Potential geotechnical and structural failure modes identified and evaluated since that time can lead to flooding risks and impacts at lower elevations than overtopping, thus increasing the benefits. Similarly, the relative project costs are greater due to the measures required to address these additional project concerns. The addition of new potential failure modes, and the plan formulation to address them, represents a change in the Future With Project condition upon which the initial economic analysis was based. A new comparison of the costs and benefits of the different levee height alternatives was required to verify the optimization of net benefits.

Updated economic benefits were determined for the KR1 and KR2 raises. A review of the Recommended Plan cost estimate was conducted to determine costs for the lower raise alternatives. As the different alternatives are in the same locations, requiring essentially the same easements, equipment, contracting, design effort, etc., there is only a relatively small cost savings of building a levee one or two feet lower. The primary cost differences are related to material quantities of earth and concrete for the levee and floodwall raises and underseepage berms, and the number of required relief wells. An update to Table 4-1 is presented in Table 4-4. As shown in the table, each individual unit, and the three-unit Kansas River system collectively, continue to show rising net benefits at the KR3 raise alternative.

Table 4-4: Updated Screening Analysis of Alternative Raise Profiles

Kansas River Plan 1 (KR1)					
Unit	**First Cost**	**Total Annual Cost**	**Total Annual Benefits**	**B/C Ratio**	**Net Benefits**
Argentine	$59,812.5	$3,279.3	$17,367.1	5.30	$14,087.8
Armourdale	$219,948.0	$12,428.9	$47,685.6	3.84	$35,256.6
CID-KS	$74,135.0	$4,190.1	$5,430.4	1.30	$1,240.3
Total	$353,895.5	$19,898.3	$70,483.1	3.54	$50,584.8
Kansas River Plan 2 (KR2)					
Unit	**First Cost**	**Total Annual Cost**	**Total Annual Benefits**	**B/C Ratio**	**Net Benefits**
Argentine	$61,446.8	$3,368.5	$17,620.7	5.23	$14,252.2
Armourdale	$223,814.0	$12,640.0	$48,465.7	3.83	$35,825.7
CID-KS	$81,157.0	$4,573.5	$6,532.1	1.43	$1,958.7
Total	$366,417.8	$20,582.0	$72,618.6	3.53	$52,036.5
Kansas River Plan 3 (KR3)					
Unit	**First Cost**	**Total Annual Cost**	**Total Annual Benefits**	**B/C Ratio**	**Net Benefits**
Argentine	$63,923.7	$3,503.8	$18,175.2	5.19	$14,671.4
Armourdale	$232,984.0	$13,140.8	$50,006.8	3.81	$36,866.1
CID-KS	$83,682.0	$4,711.4	$7,389.0	1.57	$2,677.7
Total	$380,589.7	$21,355.9	$75,571.1	3.54	$54,215.2

Notes: Oct 2012 prices; 3.75% interest rate; $000s

4.6.2 Recommended Plan Cost Estimate and Cost Risk

Project costs are summarized in Table 4-5. For additional detail of the cost estimates and cost risk analysis, see the Cost Estimating Appendix. Costs were prepared by cost engineering for each of the alternatives. All costs include interest during construction computations which assume project completion in mid-2026. All costs reflect an October 2013 price level and the annualized totals reflect the current Federal interest rate of 3.5 percent and a 50-year period of analysis. OMRR&R costs were included in this analysis for those features that will incur a net cost over and above present levels. The additional OMRR&R is due to net increases of twenty relief wells in the CID unit and fifty-nine in the Armourdale unit.

Table 4-5: Project Cost Summary

Category	**Cost ($1,000's)**			
	Armourdale	**CID-KS**	**CID-MO**	**Total**
Lands & Damages	$ 2,024	$ 1,730	$ 0	$ 3,754
Construction Elements				
Relocations	$ 1,389	$ 246	$ 0	$ 1,635
Floodwalls and Levees	$ 145,867	$ 49,451	$ 380	$ 195,698
Pumping Plants	$ 5,943	$ 1,971	$ 0	$ 7,914
Subtotal	$ 153,199	$ 51,668	$ 380	$ 205,247
Preconstruction, Engineering, and Design (PED)	$ 11,934	$ 4,156	$ 32	$ 16,122
Construction Management	$ 10,724	$ 3,616	$ 27	$ 14,367
Contingencies	$ 54,769	$ 19,006	$ 136	$ 73,912
Total First Cost	$ 232,650	$ 80,177	$ 575	$ 313,402
Interest During Construction (IDC)	$ 52,388.5	$18,361.2	$127.3	$ 70,877.0
OMRR&R	$ 191.6	$ 144.9	$ 0	$ 336.5
Total Annual Costs	**$ 12,343.80**	**$ 4,345.90**	**$ 29.90**	**$ 16,719.70**

Total first costs = PED + LERRD + construction + S&A
Annual costs = ((Total first costs + IDC) x I&A factor of 0.004457) + OMRR&R
Annual OMRR&R costs include only additional costs over and above existing costs

For each unit a cost and schedule risk analysis was conducted that identified possible risks, their likelihood of occurrence, and the significance of their impact. A Monte Carlo computer model then calculated multiple iterations and combinations of the possible risks and resulted in the appropriate contingency percentage applied to each estimate to ensure an 80% confidence that the recommended plan will not exceed the estimated cost. The majority of the risks driving the contingency are unrelated to the technical issues of the study and thus much more difficult to control. The risks showing the highest impact to the contingency are:

- Market/Bidding Conditions. The economy is in a downturn. Contractors looking for work will compete aggressively for large jobs.

- Adequacy of Project Funding. Estimate and project schedule assumes optimal availability of funds. Risk considers both incremental congressional appropriations and the Sponsors ability to cost share. Slow funding extends project schedule resulting in higher costs for future inflation.

- Undefined Acquisition Strategy. Project estimate assumes full and open competitive bidding for contract acquisition. Changes in acquisition strategy may affect competition costs.

- Contract Modifications. Unknown or unforeseen site conditions or changes not currently captured in the cost that will require contract modifications.

- Prime/Subcontractor Structure. Estimate assumes large business competitive bids. More subcontracting increases overhead and markups.

- Confidence in Scope. In some cases plan formulation was made on limited information, leading to assumptions by the designers. Risk factors were assigned to specific pieces of the Recommend Plan scope.

  - CID. Risk factor assumes possible change from floodwall modification to partial floodwall replacement and possibility of one additional pump station required

  - Armourdale. Risk factor assumes an increase in relief wells required, changes in the cost of gatewell modifications, and a possible change in the length of floodwalls needing replacement.

4.6.3 Recommended Plan Economic Analysis

Economic analysis discussed previously identified the expected economic impact of future flooding with the existing project. To aid in comparison of the alternatives, additional economic analysis was conducted to develop a risk-based evaluation in terms of benefits, costs, and performance of the alternatives under the with-project condition. The analysis encompasses all flood-prone properties within the study area.

Extensive economic surveys of the whole Kansas Citys Levees study area were completed in 2002. Economic data developed for this analysis includes values, elevations and depth-damage relationships for homes, businesses, public facilities, roads, and railroads in the study area. Furthermore, a follow up survey was conducted in early FY2012 to update the economic field data. Conditions are evaluated in terms of a base year of 2026 when the project would be operational and a future without-project conditions year of 2049. The same data set was used for both 2026 and 2049 conditions.

A risk-based economic damage analysis was performed using the HEC-FDA software that is standard in the Corps for flood damage reduction analyses. Water surface profiles with stages and discharges were obtained for eight probability events: 0.10, 0.01, 0.005, 0.002, 0.0013, .0001, 0.0008, and 0.0007. The profiles are referenced to 2008 conditions, although it should be noted that no increases in these stages are forecasted through the period of analysis and the same profiles are used for existing, base year, and future conditions. The exceedance-probability relationship for the Kansas River was evaluated using the graphical method, which involves specifying a discharge-probability relationship (including a discharge for the 0.999 probability event) for each index point along with the equivalent record length for the stream. Top of levee stages based on the critical levee low point were translated to each index point, as were exterior-interior stage relationships. Geotechnical and structural probability of failure curves were developed for critical sections on each levee, adjusted to the appropriate index points, and a combined probability of failure was computed using a formula from ETL 1110-2-556, *Risk Based Analysis for Geotechnical Engineering for Support of Planning Studies* (Formula: Pr(f) = 1-(1-p1)(1-p2),,,(1-pn)). The resulting combined probability of failure versus river stage curve was entered into the HEC-FDA study file in the "Levee Features" section.

It can be seen in Table 4-6 that in addition to the strong benefit-cost ratio for the Kansas River system-wide project, each unit is also individually justified. The combined Phase 2 portion of the total project has a benefit-cost ratio of 3.4, while Armourdale unit's benefit-cost ratio is 4.1 and the CID portion stands at 1.2. With Phase 2 net benefits of $39.5 million, the project represents a strong contribution to national economic outputs.

Table 4-6: Economic Analysis Summary

Oct 2013 prices; 3.5% interest rate; 50 year period of analysis; $1000s

Levee Unit Alternative	Annual Costs	Annual Benefits	Benefit-Cost Ratio	Net Benefits
Armourdale				
KR3 Plan	$ 12,343.8	$ 51,457.1	4.2	$ 39,113.2
Central Industrial District				
KR3 Plan	$ 4,375.9	$ 5,229.6	1.2	$ 853.7
Total Phase 2 Study Area	**$ 16,719.7**	**$ 56,686.6**	**3.4**	**$ 39,966.9**
Authorized Argentine Plan	$ 3,821.5	$ 18,180.0	4.8	$ 14,358.5
Kansas River System	**$ 20,541.2**	**$ 74,866.6**	**3.6**	**$ 54,325.4**

The primary benefits of the Recommended Plan are the reductions in the potential for flood damage. Because much of the protected area is already industrial, implementation of the Recommended Plan will provide continuity to the current employment base of the area. In the long-term, business volume, personal income, employment, and taxes are not expected to change significantly as a result of implementing the Recommended Plan. However, with improved flood risk management, new business and investment would be more easily attracted to the protected area if vacancies were to occur.

During the short-term, construction of the Recommended Plan can be expected to temporarily increase employment. The temporary presence of construction workers is likely to being a temporary increase in the demand for local area goods and services. Taken together, this is likely to result in a temporary increase in retail business and associated profits, and increased sales tax receipts at the local level.

4.6.4 Principles of Flood Risk Management Planning and Associated Analysis

The Corps of Engineers functions and operates in accordance with laws established by Congress. The Corps develops policy and guidance for implementation of the laws under which it operates. The laws, and Corps policy and guidance, provide for the use of prescribed methodologies and nationwide uniformity in the Corps planning process. Corps planning products are reviewed locally, independently, and by three levels of Washington review, i.e., Corps Headquarters, Assistant Secretary of the Army for Civil Works, and Office of Management and Budget. Reviews not only ensure consistency and accuracy in the application of the prescribed methodologies, but determine and confirm that the work was completed with adherence to guidance, policy and the law.

The structured and uniform planning process implemented and followed by the Corps of Engineers is documented in Engineering Regulation 1105-2-100, Planning Guidance Notebook. This regulation is grounded in the laws which apply to the Civil Works Program and to the Corps of Engineers missions, and is particularly based on the Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies (P&G) (March 10, 1983). The P&G were established pursuant to Section 103 of the Water Resources Planning Act (Public Law 89-80) and Executive Order 11747.

Corps policy and guidance provide for proper and consistent planning in the formulation of reasonable plans responsive to National, State, and local concerns. The resulting plans recommended for implementation are economically and environmentally sound and in general reasonably maximize net national economic development benefits, consistent with protecting the Nation's environment (NED plan). Contributions to national economic development (NED) are increases in the net value of the national output of goods and services, and are the direct net benefits that accrue in the planning area and in the rest of the nation as a result of project implementation.

The Corps uniform planning process includes certain fundamental principles in the analysis of flood risk management alternatives. These principles include, among others:

- With and Without-Project Analysis. The without-project condition is the most likely condition expected to exist in the future in the absence of a proposed water resources project. The future without project condition constitutes the benchmark against which plans are evaluated.

- Benefit-Cost Analysis and Cost Effectiveness Analysis. This is a framework used in evaluating government investments. All pertinent costs and effects of a proposed project are systematically identified and tallied. The stream of monetized benefits that occur through time with project implementation are accumulated and are discounted to a base year in order to express a single total benefit figure. Similarly on the cost side the same accumulating and discounting process is conducted so the costs are also expressed as a single value in the base year. This process allows direct comparison of benefits and costs on a common basis. If the benefits exceed the costs the project is considered economically justified. Allowable benefits categories and required cost categories to be used in analysis of Corps water resource projects are standardized across the nation. Cost effectiveness analysis seeks to answer the question: given an adequately described objective, what is the least-costly way of attaining that objective.

- Net Benefits, Optimization Analysis. Benefits can be monetary or non-monetary. The scale of flood risk management alternative that reasonably maximizes expected net benefits (returns the greatest excess of benefits over costs) is the National Economic Development (NED) Plan.

- Risk and Uncertainty. Risk-based analysis is defined as an approach to evaluation and decision making that explicitly, and to the extent practical, analytically, incorporates considerations of risk and uncertainty in a flood risk management study. In water resources planning, risk-based analysis is used to compare plans in terms of the likelihood and variability of their physical performance, economic success, and residual risks. It captures and quantifies the extent of risk and uncertainty in the various planning and design components of an investment project.

4.6.5 Risk Based Analysis of Flood Risk Management Alternatives

Flood risk management projects can significantly reduce risk of flooding, but 100% absolute protection from flooding is not an achievable goal. A zero residual risk does not exist because no project can completely eliminate natural hazards. Flooding may occur less frequently but there is always some residual risk of flooding after implementation of any flood risk management project.

Historically, many flood control projects were planned, designed, and constructed on the Standard Project Flood (SPF). The SPF was generated using modeling techniques to determine a single target design discharge. In later years, the SPF may have been associated with a return interval to describe an expected level of protection for a given flood control project. In the context of risk analysis guidance, the SPF is no longer used for a "target design". Instead, a range of floods, including those that exceed the SPF, are to be used in formulation and evaluation of alternatives. The historic SPF method relied on safety factors and freeboard, estimates of worst case scenarios, and other indirect methods to compensate for uncertainty. These indirect methods were necessitated due to the mathematical complexities involved in computing the interaction of uncertainties in hydrologic, hydraulic, and economic functions. However, with computational advances it is now possible to describe these uncertainties explicitly and calculate that interaction.

For risk and uncertainty analysis, the Corps of Engineers uses risk-based analysis procedures for formulating and evaluating flood risk management measures according to guidance in Engineering Manual 1110-2-1619, Engineering and Design Risk Based Analysis for Flood Damage Reduction Studies; and in Engineering Regulation 1105-2-101, Planning Risk Analysis for Flood Damage Reduction Studies. Risk and uncertainty arise from measurement errors and from the underlying variability of complex natural, social, and economic situations. Flooding is random in nature and flood problems are multi-dimensional making it difficult to fully understand, document, and model the physical nature of flooding, its magnitude, its probability of occurrence, and its consequences. Risk is defined as the probability an area will be flooded, resulting in undesirable consequences. Uncertainty is a measure of imprecision of knowledge of parameters and functions used to describe the hydraulic, hydrologic, geotechnical, structural, and economic aspects of a project plan.

In water resource planning for flood risk management, uncertainties in the hydrologic and hydraulic data about discharges and flood stages, uncertainties in economic data about investment values, beginning damage elevations, and damages with various flood depths, and uncertainties about the potential for geotechnical or structural failure of features in an existing flood control project can have significant impact on the residual damages, benefits, costs, planning, design, and reliabilities of a proposed flood control project.

To develop a risk based analysis as required by regulation, the Corps uses the HEC (Hydrologic Engineering Center) Flood Damage Analysis (HEC-FDA) model. The HEC-FDA model combines the engineering and economic study data to determine economic performance (flood damages) and engineering performance (probability of design exceedance) with and without a flood control project. The HEC-FDA model uses the Monte Carlo simulation process which incorporates the risk and uncertainties associated with the required HEC-FDA input values.

Planners cannot know with full certainty the exact value of a variable that may ultimately be important to the selection and implementation of a plan. The analysis instead considers a best estimate of the value, and recognizes the uncertainty inherent in that value by also using other possible values (often in terms of input curve). The range of outcomes in some areas of risk and uncertainty can be reasonably described or characterized by a probability distribution. Certain future demographic, economic, hydrologic, and meteorological events are essentially unpredictable because they are subject to random influences; however the randomness can sometimes be described by a probability distribution based on historical data. If there is no historical database, the probability distribution of random future events can be described subjectively, based on insight and judgment.

Key variables explicitly incorporated into the risk based analyses used in the Kansas Citys feasibility study included the following:

Hydraulic uncertainty. A stage-exceedance probability function was developed from the water surface profiles and a normal probability distribution was selected. Conveyance roughness and cross-section geometry were evaluated to determine a standard deviation of 1.5 feet in the base year and 1.8 feet in future years for uncertainty in river elevation, given a certain discharge.

Hydrologic uncertainty. A graphical discharge-frequency exceedance probability function was developed in the HEC-FDA model for each reach based on a 70 year period of record. The distribution of errors is assumed to be a non-central t-distribution about the specified function.

Investment value uncertainty. Interview data about most likely structure and content values, and the minimum and maximum range of values for each were obtained from business owners and representatives and entered into HEC-FDA. For structures that did not have specific data obtained by surveys and interviews, expected values for structures and contents were estimated using Marshall & Swift professional valuation software or from locally obtained study area data for similar businesses. The uncertainty was defined using a normal or triangular probability distribution, depending on the type of structure and category of damage, and any other specific data available.

Structure and beginning damage elevation uncertainty. Uncertainties about ground and first floor elevations (beginning damage elevations) were determined based on two and four foot contours on study area mapping. Uncertainties were determined per guidance in Engineering Manual 1110-2-1619, Risk Based Analysis for Flood Damage Reduction Studies.

Depth-damage relationship uncertainty. Structure occupancy types were defined for each type of structure and category of damage. The structure occupancy code defines the depth-percent damage function and its uncertainties. Normal and triangular probability distributions were used based on the category of damage, type of structure, and type of use.

Uncertainty about geotechnical or structural failure. Probabilities of geotechnical and structural failure in each unit were developed using engineering analysis. Geotechnical and structural engineers determined the most likely expected modes and sites of failure prior to overtopping in each unit. A range of conditional probabilities of failure versus river stage elevation encompassing the probable failure point and non failure point were determined for each site/mode of failure. The river elevation versus probability of failure relationship developed by the geotechnical and structural engineers for each potential failure site/mode was then translated to the index point of the reach (levee unit) and each individual potential failure site/mode was determined to be independent. The probabilities of failure for each site/mode were then combined using a formula contained in ETL 1110-2-556 to derive a single combined probability of failure versus river stage curve that accounted for all the sites or modes of potential failure. The resulting combined probability of failure curve was then entered into the HECFDA study file.

4.6.6 Other Considerations Related to Risk and Reliability

It is important to bear in mind the variability and uncertainty associated with the inputs to a risk and uncertainty analysis.

- Care must be taken to consider the entire output of the analysis rather than placing undue reliance on any one statistic.
- Such simulations are sensitive to assumptions about correlations between parameters, the likelihood that a particular specification is correct, any omitted factors, and assumptions about the appropriate distribution for parameters, etc.
- Generally, the quality of the overall analysis is reflective of the quality (or accuracy) of its input components.

This final feasibility study is, in many respects, a groundbreaking effort with regard to the scale and scope of effort. In the past, many Corps studies have been performed using risk and uncertainty principles for planning smaller levee systems limited to flood events at or about the 1% event. The target conveyance in the original authorizations places this system in the upper echelon of U.S. levee systems. This makes it difficult for direct comparisons to other levee systems of the results and reliabilities produced by this analysis. The possibility for better characterization and comparison for residual risk is expected as the number of larger levee systems analyzed using risk and uncertainty principles increases over time.

In general, water resource development and planning continues to be a field where judgment and context plays a vital role. There can never be one exact solution to all conceivable issues. The feasibility process undertaken in this study allows for a reasoned and systematic approach to formulating plans. However, natural environments and especially the dynamic characteristics inherent in river systems, remain subject to re-interpretation and refinements as the knowledge base and experience with those systems grow over time.

4.6.7 Selection of the Recommended Plan

When evaluating alternative levee raises, incremental economic analysis strongly affects the optimization and selection process. Levee raise costs typically increase as the levee height increases. These cost increases arise from the various components of cost that increase along with levee height: additional material and construction requirements, additional real estate costs, and a longer construction period (Interest During Construction). Other life cycle costs (such as operation and maintenance costs over the period of analysis) are included in the analysis. The optimal raise is the one with the greatest net economic benefits (essentially damages reduced less project economic costs) as computed for an array of flood events. As the evaluation progressed, Kansas River Plan 3 (KR3) was shown to be an efficient raise meeting the planning objectives, constraints, and criteria; maximizing the net economic benefits; limiting land disturbance and environmental impacts; and avoiding HTRW disturbances and significant real estate conflicts and relocations. Plan KR3 is the Recommended Plan.

All features of the individual Recommended Plans identified for CID and Armourdale were retained and combined to constitute the Recommended Plan for the Phase 2 study area. The plan grows from an assembly of the most technically feasible and cost effective measures to achieve the desired unit height in each reach and/or area of concern in each unit. The economic analysis shows that it is economically viable and furthers national economic development in manner consistent with Corps of Engineers economic regulations and Administration economic policies.

It should be made clear that while this Recommended Plan addresses existing concerns and improves the risk management benefits provided by the existing units, consistent with the rest of the Kansas Citys Levee System, it does not provide a system capable of passing the authorized Kansas River discharge. Modifications to comply with the authorized discharge for the Armourdale and CID Units, given the results and findings of the current engineering analysis, would very likely be uneconomical or unaffordable, and would be inconsistent with the rest of the system, causing induced damages or risks to other units of the metropolitan system if implemented.

The tax bases within both of the levee units are relatively stable as the protected areas are essentially built-out. This limitation on tax base essentially places an upper limit on the potential for totally local initiatives. The Recommended Plan leverages local funding through the Federal cost share process. It is likely that several of the major recommendations herein would remain un-built if not for the Federal cost sharing opportunity provided by the Recommended Plan. The Recommended Plan also provides many lower income residents with additional flood risk management benefits which might not otherwise be available through local processes.

4.7 Description of the Recommended Plan

4.7.1 Components of the Recommended Plan

The major components of the Recommended Plan are summarized in Table 4-7. A comparison of the current and recommended features by levee station is provided in Exhibits 5 and 6. Maps of the Recommended Plan are provided following the main report text.

Table 4-7: Recommended Plan Components

Overtopping/Structural Measures	CID	Armourdale	Total
Levee Raise (LF)	6,495	13,223	19,718
Floodwall Modification(LF)	4,649	4,208	8,857
Floodwall Replacement (LF)	152	2,105	2,257
New Floodwall (LF)	600	5,392	5,992
New T-Wall on Levee (LF)	-	7,715	7,715
Closure Structure Measures			
New Sandbag Closure	2	3	5
Convert Sandbag to Stop log	1	2	3
Replace Stop log Closure	1	2	3
New Stop log Closure	2	-	2
Underseepage Control Measures			
New Relief Wells	57	74	131
Area Fill (LF)	3,448	-	3,448
Slurry Cutoff Wall (LF)	-	2,000	2,000
Drainage Control Measures			
Pump Station Removal	2	2	4
Pump Station Modification	5	7	12

4.7.2 Summary of Recommended Plan Performance

The with-project (residual) flood risks and damages are shown in Table 4-8. The residual risk results address all three major aspects of the levee performance analysis: overtopping (hydraulic), geotechnical, and structural. The with-project performance provides a very significant decrease in the flood risk for each of the respective units.

Table 4-8: Engineering Performance - With Project Conditions

	Armourdale	CID
Annual Exceedance Probability* (median)	0.12%	0.12%
Annual Exceedance Probability* (expected)	0.14%	0.19%
Long Term Risk (chance of exceedance during indicated period)		
over 10 years	1.39%	1.84%
over 30 years	4.10%	5.43%
over 50 years	6.74%	8.88%
Conditional Exceedance Probability** - Overtop or Breach		
10.0% event	0.00%	0.03%
4.0% event	0.00%	0.03%
2.0% event	0.04%	0.03%
1.0% event	1.39%	0.73%
0.4% event	14.51%	10.17%
0.2% event	34.79%	28.88%

Notes:
*Annual exceedance probability is the chance of experiencing any flood event - of whatever magnitude - within any year.
**Conditional exceedance probability is the probability that specified flood event would overtop or breach the levee.

Through implementation of the Recommended Plan, both levee units will be designed and constructed to meet current USACE requirements for a positive levee evaluation. Furthermore, each unit will comply with FEMA base flood insurance certification and accreditation requirements, including passing the 1% event with at least 90% assurance. Both units will have approximately 65-70% assurance against the median 0.2% chance exceedance flood profile. Other performance aspects of the with-project condition are described in additional detail below.

4.7.3 Future With-and Without-Project Condition Economic Performance

Implementation of the Recommended Plan in each of the units addressed in the final feasibility report will provide significant reduction in physical flood damages and other costs that result from flooding. The damages reduced represent the benefits provided by the recommended plan and are typically characterized in terms of annualized equivalent values as computed in the HEC-FDA program.

Table 4-9 summarizes the equivalent annual damages that would be expected to occur with and without the recommended plan. The uncertainties in evaluation of project benefits are characterized in the far right three columns of the table.

Table 4-9: Equivalent Annual Damages and Damages Reduced
(Oct 2013 Prices, 3.5% Inter Rate, 50 Yr Period of Analysis, $000)

Plan	Top of Levee Elev. (ft)	Expected and Probabilistic Values of EAD and EAD Reduced					
		Equivalent Annual Damage			Probability Damage Reduced Exceeds Indicated Values		
		Total Without Project	Total With Project	Damage Reduced	.75	.50	.25
ARMOURDALE UNIT							
Future **WITHOUT** Project	771.70	$55,392.04	$55,392.04	$0.00	$0.00	$0.00	$0.00
Future **WITH** Project	776.41	$55,392.04	$3,395.04	$51,457.05	$36,287.34	$49,899.39	$63,998.28
CENTRAL INDUSTRIAL DISTRICT							
Future **WITHOUT** Project	760.30	$8,867.90	$8,867.90	$0.00	$0.00	$0.00	$0.00
Future **WITH** Project	763.45	$8,867.90	$3,638.32	$5,229.58	$1,583.39	$3,769.20	$7,442.57

4.7.4 Future With- and Without-Project Condition Engineering Performance

4.7.4.1 *Conditional Probability of Design Non-Exceedance.*

One of the many metrics that can be used to characterize the performance of a flood risk management project is overall project reliability against the 1% event. Project reliability is characterized in the HEC-FDA model by the probability of the project design containing a specified event or the probability of design non-exceedance. Overall reliability against the 1% event and other engineering performance data include consideration of both the probability of overtopping and also the probability of geotechnical and structural failure.

Table 4-10 displays for each unit addressed in the Final Feasibility Report the with- and without-project condition overall project reliability against the 1% probability event, and shows the top of levee margins above the 1% and 0.2% event water surface profile.

4.7.4.2 ***Levee Performance in Any Given Year and Equivalent Long-term Risk***
Long-term risk indicates how successfully a flood control project would protect against floods given the uncertainties and over a long period of time. Annual Exceedance Probability is the probability that flooding will occur in any given year considering the full range of possible annual floods. The terms "exceeded" or "exceedance" when used herein with regard to engineering performance data include consideration of both geotechnical and structural failure potential and consideration of the potential for levee overtopping.

Table 4-11 shows the expected probability of the levee design being exceeded (occurrence of flooding) in any given year and the long-term risk or probability of the project being exceeded in a 10-, 30-, and 50-year period, with and without the recommended plan for each unit.

Table 4-10: Future Condition Overtopping Margins and Overall Reliability Against the 1% Chance Event

	Top of Levee Elev. at Index Point (ft, msl)	**Overtopping Margin (ft) Above 1.0% Chance Event**	**Overtopping Margin (ft) Above 0.2% Chance Event**	**Overall Reliability Against 1% Chance Event (includes geotechnical and structural risk considerations)**
ARMOURDALE UNIT				
Future **WITHOUT** Project	771.70	6.89	-1.92	.4547
Future **WITH** Project	776.41	11.66	3.17	.9861
Net Change in Margins and Overall Reliability	+4.71	+4.78	+5.10	+.5314
CENTRAL INDUSTRIAL DISTRICT UNIT				
Future **WITHOUT** Project	760.30	5.06	-0.87	.8866
Future **WITH** Project	763.45	8.21	2.28	.9927
Net Change in Margins and Overall Reliability	+3.15	+3.15	+3.15	+.1061

*Any discrepancies due to rounding

Table 4-11: Recommended Plan Engineering Performance and Equivalent Long-Term Risk

	Top of Levee Elevation at Index Point (ft, msl)	Annual Exceedance Probability	Equivalent Long-Term Risk Probability of Exceedance Over the Indicated Time Period		
			10 Years	30 Years	50 Years
ARMOURDALE UNIT					
Future **WITHOUT** Project	771.70	.0350	.3148	.6784	.8490
Future **WITH** Project	776.41	.0014	.0139	.0410	.0674
Net Change in Probability of Exceedance (Flooding)	+4.71	-.0366	-.3009	-.6374	-.7816
CENTRAL INDUSTRIAL DISTRICT UNIT					
Future **WITHOUT** Project	760.30	.0047	.0461	.1321	.2103
Future **WITH** Project	763.45	.0019	.0184	.0543	.0888
Net Change in Probability of Exceedance (Flooding)	+3.15	-.0028	-.0277	-.0778	-.1215

Note: Any discrepancies due to rounding

As shown in Table 4-12, long term risk can be alternatively described in terms of chance of flooding in any one year or in a specified time period.

Table 4-12: Alternative Display of Recommended Plan Engineering Performance and Equivalent Long Term Risk

	Top of Levee Elevation at Index Point (ft msl)	Chance of Exceedance in any Given Year	Equivalent Long-Term Risk Chance of Exceedance Over the Indicated Time Period		
			10 Years	30 Years	50 Years
ARMOURDALE UNIT					
Future **WITHOUT** Project	771.70	1 in 28.6	1 in 3.2	1 in 1.6	1 in 1.2
Future **WITH** Project	776.41	1 in 833.3	1 in 71.9	1 in 24.4	1 in 14.8
CENTRAL INDUSTRIAL DISTRICT UNIT					
Future **WITHOUT** Project	760.30	1 in 303.0	1 in 21.7	1 in 9.0	1 in 4.8
Future **WITH** Project	763.45	1 in 833.3	1 in 54.3	1 in 18.4	1 in 11.3

Note: Any discrepancies due to rounding

4.7.5 Induced Damages

The Interim Feasibility Report included the following discussion of induced damages: *Minor induced damages from the Argentine levee unit raise, which can occur under certain rare and somewhat extraordinary conditions. If one of these rare flood events occurs, then minor induced damages could possibly occur in the following areas:*

- *Areas downstream of the Argentine Unit (areas within the existing Armourdale and CID Units)*
- *In a small unprotected area opposite the Armourdale Unit and located below the bluff line.*

The flood events for which these induced damages can be calculated to possibly occur are more rare than the 250 year (or 0.4%) event and approaching the 300 year (0.33%) event. In these situations the induced flooding is very small (about 6 inches deep in most cases). Given this, the induced damages amount on each structure is essentially inconsequential compared to the existing damages from normal river flooding. The predominant threat of flooding in these areas remains essentially the same as the without-raise conditions. While the events that may trigger these induced damages are rare, in accordance with economic policy the costs associated with induced damages are recognized in the study economics.

These relatively small induced damages discussed would occur only if the Argentine Unit was raised and the downstream units, Armourdale and CID, were not. The Recommended plan for raising the Armourdale and CID Units eliminates these induced damages. The Recommended Plan causes no new induced damages on other areas.

4.7.6 Residual Risk

Although floodplain users and occupants may desire total protection from flooding, it cannot be overemphasized that this is an unachievable goal. Residual Risk will remain after completion of the Recommended Plan. The primary source of residual flood risk will be from infrequent large flood events that overtop the levees. A number of factors can influence the nature of flood inducing storm events and the performance of flood risk management systems, such that an event of historical magnitude is not necessarily required to overwhelm the project and cause catastrophic damage. However, the implementation of project improvements may lead many floodplain users and occupants to feel that they have near-total protection against flooding. Therefore, it is important to emphasize and communicate the level of flood risk that remains even after project implementation such that floodplain occupants are aware of the nature of the flood threats and are able to make informed decisions about acceptable levels of risk.

The tables presented in this report show that the recommended plan for the units addressed by this final feasibility report provides a significant increase in reliability against flooding. Flooding will be less frequent; however, the analyses show there is still residual risk of flooding. For the Corps, determining an acceptable level of risk is in most cases a function of the NED process. The goal is to manage the risk of flooding within limited budget and funding constraints, and yet implement a cost effective and efficient flood risk management plan that reasonably maximizes net economic benefits (flood risk management benefits) consistent with protecting the Nation's environment.

From the Federal perspective, selection of the recommended alternative is a determination of an acceptable level of residual risk based on trade-offs between potential benefits and the associated level of residual risk versus the cost of a larger and more risk-adverse flood risk management project. Increases in project reliability above what is recommended can sometimes be achieved with much larger projects. However, in most instances, costs for larger projects increase dramatically faster than project benefits. The Recommended Plan reasonably maximizes net benefits consistent with study objectives and constraints as measured by the difference between annual benefits and annual costs.

From the local perspective, a community or sponsor may desire less residual risk of flooding than that provided by the Recommended Plan. Many persons in a community might express the desire for zero residual risk and no chance of damage from a recurrence of flooding, even though this is an economically unattainable goal. The level of risk a community (or sponsor) is willing to bear can be indicated by their willingness to pay for each additional increment of flood risk reduction.

4.7.6.1 ***With-Project Damages and Impacts***

The selected plan has substantial economic benefits and reduces the overall study area equivalent annual damages in the existing condition by nearly 88% (93% in Armourdale and 59% in the CID). The probability and occurrence of flooding will be greatly diminished. There would remain a significant total of residual equivalent annual damages of $7.57 million ($3.93 million in Armourdale and $3.64 million in CID).

Tables 3-11 and 4-8 compare the existing and future-with-project assurance statistics for the two levee units. Comparing the expected annual exceedance probabilities there remains a 0.14 and 0.19 percent chance of a damaging flood in Armourdale and CID, respectively, in any year following project implementation. In the 1 percent-chance flood event, both Phase 2 units currently have between an 11 and 55 percent chance of experiencing damage due to overtopping or breach failure. These probabilities would be reduced to roughly 1 percent in the with-project condition. The remaining risk of incurring damage at the 0.2 percent-chance flood event over the same time period is approximately 32%. The long-term risk of a damaging flood in both of the Phase 2 units over 50-year period would be less than 1 in 10, compared to a current 50-year risk exceeding 1 in 2 in Armourdale and approximately 1 in 5 in CID. While the improvements proposed are substantial, it can be seen that residual risks remain.

If the capacity of the Federal levee system is exceeded in a particular event, most of the areas and properties inside the levees would be affected due to the flat floodplain topography in these areas. The Armourdale and CID areas are generally small volumetrically in relationship to the Kansas River hydrograph. Analysis has shown that the areas would fill quickly on the rising limb of the hydrograph by the time maximum overtopping depth is reached. In general, if the amount of water that gets through or over the levees is sufficient to produce severe flood depths, event specific damages in the study area would reach $2 billion or more. Prohibitive depths of water would likely remain inside the levees for several weeks. Large-scale evacuations of urban neighborhoods would be necessary in advance, followed by humanitarian assistance. A number of highly-traveled highways and streets as well as railroad tracks would be closed and in some cases inundated. Public utilities including power generation and wastewater treatment would be interrupted, perhaps for a few weeks.

4.7.6.2 ***Life Safety Risk Assessment***

The Corps of Engineers Levee Safety Program evaluates a number of safety criteria using the Levee Screening Tool. The LST provides a common basis on which to rate the condition and failure consequences for levees across the Nation. The LST also provides an analysis of the loss of life that could occur due to a project breach prior to overtopping (PTOT) and due to an overtopping (OT) breach. The existing condition life safety assessment evaluation for the Armourdale and CID Units is shown in Table 4-13.

Table 4-13 – Existing Condition Life Safety Assessment

	Population at Risk		Threatened Population		Loss of Life	
	Day	Night	Day	Night	Breach PTOT	OT Breach
Armourdale	6,700	2,924	1,817	681	19	9
CID	7,274	813	2,503	252	22	14
Total	13,974	3,737	4,320	933	41	23

Population at Risk (PAR) is representative of the occupants and users within the levee units. The Recommended Plan will not cause any changes (upwards or downwards) in the Population at Risk. The study area is already fully developed, and improvements to the levee systems will not promote additional development nor does the Recommended Plan change the area limits. Threatened population is an estimate of that portion of the PAR that would still be remaining in the floodplain at the time of project failure. The resulting estimate of Loss of Life is heavily influenced by the determination of threatened population; however, both are influenced by other factors including:

1. Probability of overtopping failure
2. Project reliability at events below top of levee
3. Unit geometry and inundation characteristics
4. Quality of emergency planning and risk communication prior to project failure

The Recommended Plan directly addresses the first two of these factors by increasing reliability of project features and reducing overtopping probability. This will allow the occupants of the floodplain more time to implement emergency procedures and evacuations, if needed, potentially decreasing the threatened population and loss of life.

The Recommended Plan also affects unit geometry and inundation characteristics. Raising the height of the existing unit reduces the likelihood and frequency of inundation and can lower the total loss of life expected during the life of the project. However, a higher levee unit will cause increased inundation depths during an actual project failure, which could increase the potential for loss of life during that event. This is a lesser concern in the CID Unit, where floodwaters entering from the Kansas River would be expected to exit over or through the Missouri River section of the unit, which is not being raised.

Emergency planning and communication is the biggest driver of threatened population and loss of life analysis. If the occupants of the floodplain are well informed of the risks and emergency procedures in advance, and are able and willing to implement those actions when directed, including compliance with evacuation orders, the loss of life can be significantly reduced. The Feasibility Study process included public information and involvement, which helped to inform the public of the risks, but the Recommended Plan contains no components specific to emergency planning or communication.

Because of the different factors that can affect loss of life estimates, and the dependency of life safety concerns on actions beyond those addressed by the Recommended Plan, no attempt has been made to correlate plan implementation to a reduction in loss of life.

4.7.6.3 ***Residual Risk Management***

Informed risk management and emergency preparedness, by both the sponsor and the Corps of Engineers, is the manner in which residual risks and potential exceedance of the system will be addressed. Based on the hydraulic analysis of the Kansas River units it is expected that overtopping would begin at or near the upstream end of each individual unit during a Kansas River flood. Conversely, these units would likely overtop near the downstream end during a Missouri River flood, as was the threat in 1993. There is no advantage or evident solution in managed overtopping, i.e. designing for a specific overtopping location, in an interrelated system of levees with intensive development throughout each protected area. Effective emergency planning in advance is the best way to protect communities and minimize the damage from these rare flood events.

Each of the five sponsors within the existing system operates their unit(s) according to unit specific Operations and Maintenance Manuals originally prepared by the Corps of Engineers. Each manual contains a list of specific actions to be taken by that sponsor during emergency flood operations. The emergency actions detailed within all of the manuals are triggered by the Missouri River stage as reported on the official USGS Missouri River at Kansas City gauge. The gauge is attached to the Hannibal Railroad Bridge downstream of the confluence of the two rivers. By using a single control point, the group of manuals, and the actions of each individual sponsor, is tied together into a complete system emergency operations plan. Forecasts and warnings for the Kansas City gauge, and other gauge locations on both rivers upstream of Kansas City, are issued regularly by the National Weather Service. These forecasts include projected river flows and stages several days in advance. During normal operations these forecasts are issued daily and during flood emergencies, three times a day.

The Corps of Engineers employs a very proactive approach to monitoring and inspecting the system units, provides training for flood preparedness and flood fighting, and activates a comprehensive Emergency Operations Center (EOC), including liaison and technical assistance as needed to assist local entities in their flood response and operation of the system. During flood operations the EOC conducts a daily conference call with sponsors and stakeholders throughout the impacted area, whether in Kansas City or beyond, to disseminate and communicate all available flood status and risk information. The Kansas City Water

Management Branch and the Northwestern Division Reservoir Control Center in Omaha, NE, are regular participants in these calls and provide updates on upstream reservoir conditions and operations, and their potential impact to expected flows.

Similarly, the sponsors have monitoring, emergency response, and evacuation plans that are coordinated between the Kaw Valley Drainage District, the Kansas City, Kansas, Emergency Management Office, the City of Kansas City, Missouri flood / emergency response elements, and the business and residential areas protected by the levees. These tie together in a proactive and coordinated flood response and risk management framework with the Corps of Engineers, both in preparation and training activities as well as during flood response. Further, as assisted through this study, the Sponsors are in development of a system-wide Floodplain Management Plan that will make recommendations in an improved framework for local cooperation and risk management.

Following implementation of the Recommended Plan in each unit, the Corps of Engineers will update each O&M Manual to reflect the new with-project conditions and features, including any changes to the emergency actions list that may be needed. Each sponsor and local municipality within the study will need to modify any other emergency action, evacuation, or floodplain plans they currently have, or design new plans, to further manage and minimize the residual risks remaining after Recommended Plan implementation.

During this feasibility study effort, all sponsors in the system began an effort to develop a coordinated system-wide floodplain management plan as a combination and expansion of their existing individual plans. Sponsors will continue to this effort as this project enters PED. Those efforts will reduce potential loss of life by resulting in improvements to: evacuation planning, flood warning effectiveness, and community awareness.

4.7.7 Real Estate Requirements

Project purposes would require the expansion of the current Kaw Valley Drainage District and City of Kansas City, Missouri, easements. Required estates include temporary easements, permanent easements and borrow easements. It is estimated that roughly 33 acres will be needed for Armourdale levee raises and roughly 62 acres will be needed for CID levee raises. There are three utilities requiring relocations that have been identified as eligible for compensation on the Armourdale portion, and six on the CID portion. The estimated total amount for all creditable Lands, Easements, Rights-of-Way, Relocations, and Disposal Areas (LERRD) costs, including contingencies, for the Armourdale Unit is $4,463,000, and for the CID Unit is $2,591,000. Important aspects of the LERRD's required for the Recommended Plan are highlighted below. See the Real Estate Appendix for additional detailed information.

4.7.7.1 *Lands and Damages Costs*

For both units in the Recommended Plan, this LERRD category includes the costs for Non-Federal sponsor acquisition of lands in fee title, permanent right-of-way, temporary right-of-way; and associated and incidental costs for legal work, title work, tract appraisals, and land surveys. The recommended plan does not include any costs to address encroachments into the existing project right of way. Addressing these encroachments is the responsibility of the non-Federal sponsor.

Land acquisition anticipated for the Recommended Plan primarily consists of limited permanent and temporary easements on private and public lands. Fee acquisition is not expressly required for levee rights-of-way (r-o-w) on either of the units. Estates to be acquired by the sponsors include permanent levee and floodwall easements necessary for the levee raise (berm placement) and floodwall work.

A permanent easement will be used for a borrow area and temporary easements will be used for equipment storage, site access for construction vehicles and staging areas. Temporary access road easements will vary in width along the different work areas but are generally 15 to 30 feet wide. Duration of the temporary easements will also vary for each of the individual work areas, generally running from 1 year to 3 years. The Recommended Plan does not require acquisition of an off-site disposal area.

4.7.7.2 *Borrow Area Considerations*

The area of proposed borrow for the Armourdale and CID Unit raises is the same as previously proposed for the Argentine Unit and discussed in the Real Estate Plan included as Appendix to the Interim Feasibility Report. The borrow area was described in Section 1.2.8 of the FEIS as follows:

> The proposed borrow area measures approximately 276 acres and is owned by Water District Number One (WaterOne) of Johnson County, Kansas. The proposed borrow area is located adjacent to the right descending bank between Kansas River miles 11 and 13, Wyandotte County, Kansas. The borrow area is accessed from south 74th Street via Holliday Drive and Interstate 435. Levee access from the proposed borrow area would route from Inland Drive to South 59th Street.

The primary uses of the land are lime residual storage from the water treatment process and active row-cropping under a lease agreement, thus, existing disturbances within the proposed borrow area includes excavating, hauling, grading, and disk harrowing. WaterOne treats water from the Kansas River and occasionally the Missouri River. Because these two water sources are hard waters, WaterOne uses lime to "soften" the water by removing the carbonate hardness in the water. The by-product of this process is a lime residual that is removed from the treatment process and stored in large lagoons on this site and allowed to dry. After the drying process, which may take a few years, the dried material is excavated and removed for disposal, and the lagoons are cleared for future use. Excavated material used for the levee raises will not contain the lime residual, but the activity to obtain levee fill material from clean areas of the site will be beneficial to WaterOne in the creation of new lagoons for future lime residual storage.

Material from the borrow area will be necessary for levee raises, area fill, underseepage berms, and stability berms. The amount of fill material needed to implement the proposed levee raises in all three levee units on the Kansas River units is shown in Table 4-14.

Table 4-14: Borrow Area Requirements

Unit	Borrow Amount (*cubic yards*)
Central Industrial District	175,088
Armourdale	459,162
Argentine	261,955
Total	896,205

4.7.7.3 *Facility/Utility Relocations*
A number of existing utilities are deemed necessary to be relocated for implementation of the Recommended Plan. Utility relocations include relocations of utility crossings (crossing the raised levee) and relocations of utilities within the critical levee zone affected by increased uplift pressures. This category is further divided into: a) public utility relocation costs which are deemed compensable and are included within project LERRD, and b) those utility relocations which were deemed not compensable and are the responsibility of the utility owners (relocation of non-compensable utilities are considered an associated cost but not a project cost).

PL 91-646 relocation assistance applies to the removal and relocation costs for private business structures (less than 10,000 sf total). No costs are included for PL 91-646 assistance to business owners as no impacts of this type are expected. No residential housing is affected in any unit by the Recommended Plan.

4.7.7.4 *Transportation Facilities Impacts*
No active railroad tracks or railroad facilities require permanent relocation. Temporary adjustments to trackage or schedules are likely needed during some periods of construction. No public roads or bridge crossings require modification.

4.7.8 Design and Construction Considerations
As this study deals with an existing levee system, the site constraints arising from adjacent infrastructure must be considered during design and construction. During alternatives development and refinement, the study examined design and construction considerations important to an efficient implementation of the Recommended Plan.

In particular, work alongside rivers must consider the somewhat unpredictable nature of flood hazards. High water conditions may occur while construction is in progress. If the high water conditions were to occur while the line of protection is temporarily down or compromised by construction (such as when a floodwall is being removed), then serious inadvertent flooding could result. This situation is normally handled through the development of specific high-water contingency measures. Requirements for these contingency measures are included within the plans and specifications (construction contract) package. The construction package must address high-water contingencies for all sites in the Recommended Plan.

Such contingencies must aim to provide for at least the 1%-chance annual event as the most basic requirement. Beyond this, an additional level of preparation should be planned to bring the protection back to the preconstruction (design) level if needed under severe flood conditions.

Common site measures for water control include dewatering, construction of ring levees, and emergency backfilling of open excavations. Sandbags and pumping can also be used to supplement the effort. It is preferable to schedule work within the levee critical zone for typically dry seasons. Excavation in the levee critical zone must be avoided during periods of ground saturation.

For all sites, the project coordination team (composed of sponsors, Corps of Engineers staff, and other stakeholders deemed appropriate to the work) will take the Recommended Plan and develop the design detail and contracting documents necessary for successful construction efforts. The project management plan (PMP) will address project scope, quality, schedule, communications, safety, and project team roles as the project develops. The requirements of ER 1110-2-1150 will guide the overall design effort. The Project Partnership Agreement (PPA) will contain specific requirements regarding responsibilities, funding and coordination of construction activities. Additionally, an implementation phase Review Plan (RP) will be developed detailing the level of review each design and construction package will receive prior to award. This RP will detail the need for IEPR Type II, or Safety Assurance Review, which will include a review of all life safety concerns including emergency action planning.

The Non-Federal Sponsor will conduct specific utilities relocation coordination and design planning prior to levee raise construction contract award. Even though sponsors and utility owners are responsible for utility relocations, the Kansas City District must be kept aware of relocation designs and schedules to ensure coordination of the overall implementation effort. Detailed planning for utility relocations is fully developed in the latter stages of the PED phase in coordination with the construction plans. All parties (sponsor, utility owner, and Corps of Engineers) should prepare for a highly coordinated utility relocation effort as the levee raise begins.

In general, the following two factors will affect design and construction along several areas of the levee raise.

- Several areas along the Armourdale levee were identified as Hazardous, Toxic, or Radiological Waste (HTRW) sites. A section within the main feasibility report describes HTRW considerations of the Recommended Plan. Design and construction procedures need to recognize these sites and adapt accordingly. Construction cannot normally occur on top of contaminated soil.

- The Recommended Plan for the Armourdale and CID raises involves <u>no</u> permanent impact to existing railroad tracks, but the design and construction in for all areas with adjacent railroad tracks does require coordination with the railroads. Trains may need to be temporarily re-scheduled so as to allow movement of construction equipment into and out of the construction area.

<u>Armourdale T-wall on Levee Construction.</u> The pre-construction coordination should include careful planning sessions where the T-wall procedures are sequenced and scheduled to avoid undue delays with an open levee crown. During T-wall construction, the levee crown is removed

along with any rip rap cover. The T-wall installation proceeds and then the levee crown is rebuilt as soon as practical.

Utility Crossings. Utilities crossing the Units were studied to estimate the costs for relocation or removal of (functioning or abandoned) utilities, and for the real estate implications related to preliminary compensability determinations. As a general rule, pressure pipelines passing through or under the levee are generally relocated over the raised levee. An additional amount of earth cover tops off the utility lines and the resulting "mound" is sloped on each side to allow vehicular transverse. Normally these utility lines are hot-tapped thus maintaining service to customers during construction.

Bridges and Roadways. The Recommended Plan does not require any bridge superstructure modifications, nor does the Recommended Plan require any road realignments. Transportation of levee raise materials may at times increase traffic along nearby roadways but this area is industrial and truck traffic is common.

The final grade and slope on the raised top-of-levee access road needs close coordination with the sponsor. The raised top-of-levee road incorporates up-and-over utility crossings under the Recommended Plan. The design for these crossings points and the amount of roadway cover should allow vehicular traffic (such as passenger cars and trucks) to traverse the crossings with relative ease. The design of the top-of-levee road may need some realignment to maintain required minimum clearance under the I-635 bridge structure.

4.7.9 Operations and Maintenance Considerations

4.7.9.1 *OMRR&R Costs*

Operation, Maintenance, Repair, Replacement and Rehabilitation (OMRR&R) of the project will remain the responsibility of the non-Federal sponsors. Operation and Maintenance (O&M) manuals will be prepared (or updated as appropriate) by the Corps of Engineers and provided to the sponsors following each implementation contract or phase. Proper and timely non-Federal Sponsor operation is required to ensure the integrity and performance of the levee system as designed. Non-Federal sponsor requirements for coordination, operations, maintenance, and training, are established and governed by the existing Operations and Maintenance Manuals of each levee unit, as well as multiple existing national and local regulations and policies, and are monitored through established Corps of Engineers inspection and oversight programs.

The majority of the sponsor O&M concerns and costs will remain the same as the current condition. There will be some savings in costs related to pump station removals, although most of the stations slated for removal are already essentially abandoned and not being fully operated maintained or upgraded currently. There will be an overall net increase in the number of relief wells in the system, requiring periodic testing and rehabilitation, repairs as needed, and eventual replacements. While these relief well costs are the driver in overall changes to the O&M costs, evaluation of their impact on an annual basis indicates little overall change as shown in Table 4-15.

Table 4-15: Annual Operation & Maintenance Cost for Phase 2 Recommended Plan

Levee Sponsor	Average Annual O&M Costs	Incremental Annual O&M Cost for Recommended Plan
Kaw Valley Drainage District	$1,700,000	+$336,500
Kansas City Missouri	$875,000	+$0

Several closure structures are being converted from sandbags to stoplogs and new structures of both types are being added to the system. The necessary coordination and operational considerations of closure structures are already well understood by the sponsor and the affected stakeholders from past experience. Any changes in the recommended closure plans, i.e. notifications, timing, river elevation action levels, etc., will be documented in revisions to the Operations and Maintenance Manuals. For locations where new stoplog gaps are being placed, stoplog storage locations will be identified and the necessary easements or property requirements coordinated through the LERRD process.

4.7.9.2 *System Operations Risk*

While project Residual Risk is often expressed and understood in terms of the remaining statistical probabilities of failure after project implementation, there are factors other than the chances of flood occurrence that can contribute to the risks of poor levee system performance.

It is recognized that the five non-Federal sponsors that own, operate, and maintain the individual units within the existing system have a long history of diligent and effective management and performance. In fact, the cooperation of the sponsors in local flood risk management efforts pre-dates the involvement of the Corps of Engineers and the presently existing system. However, the separation of system operational responsibilities among multiple independent entities creates dependencies and risks unique to this system. There are several aspects to this system operations risk, including:

- If maintenance, operations, or improvement activities are not performed by all Sponsors at a level consistent with the others, the system as a whole does not perform as intended.
- The actions, or non-actions, of one entity could potentially cause increases in risks and/or damages to another.
- The multiple political boundaries (City, State, County, and Congressional) within the system create differing authorities and abilities regarding taxation, financial bonding, budget support, control of floodplain development, and the issuance and coordination of flood warnings and evacuations when needed.

The Project Sponsors are aware of these system-wide concerns and realize it is in their collective self interest to mitigate their risk by coordinating their activities whenever possible. The Sponsors conduct regular meetings and discussions among their organizations and stakeholders, and, most recently, have initiated an effort to coordinate and combine their individual floodplain management and emergency planning efforts into a single system-wide approach. The individual Sponsors are supported in many of these collaboration efforts by local municipalities and regional stakeholder groups, including the Missouri and Associated Rivers Coalition, the Mid-America Regional Council, and the Kansas City Industrial Council.

An Operation and Maintenance Manual for each levee unit addresses project specific sponsor responsibilities. Each manual spells out the specific actions necessary by the sponsor for maintenance of project features and operations of the project during normal and emergency situations (i.e. at what river levels to activate pump stations or close openings in the levee, etc.). Although each manual addresses only one specific unit, they were originally written by the Corps following initial construction of the system, and have been updated by the Corps following all major modifications. In this way the separate manuals, collectively, represent a total system operations plan. By operating their respective elements of the system according to their individual manuals, each Sponsor, insures that the system will operate as a whole, as originally intended. The manuals will be updated again by the Corps following implementation of each element of the Recommended Plan.

The responsibilities for sponsors of Federal flood risk management projects are detailed in the Code of Federal Regulations (CFR) Title 33, Part 208, as well as ER 1130-2-530 (Project Operation). It is the corresponding responsibility of the Corps to take an active role in overseeing the activities of the Sponsors and ensuring that they execute their requirements according to these policies. This enables the Corps to monitor the system as a whole, thus attenuating some of the system operational risks. For example, each levee unit is annually inspected in cooperation with each Sponsor, for compliance with maintenance requirements; all proposed construction within the vicinity of the levee system is reviewed to ensure adherence to current guidelines; and the Corps' Levee Safety Program evaluates the risk of each unit based on current condition and potential consequence. While each of these activities are conducted on a unit-by-unit basis, the same criteria, standards, and guidelines are applied to all, allowing the District to establish an overall view of the system and identify where additional assistance or emphasis may be necessary. Finally, the District's Emergency Management Branch provides periodic training and assistance on flood fighting and preparedness and provides engaged and proactive liaison, monitoring, technical, and material assistance when required during flood stage operations. This coordinated system based flood response was well tested in 1993, and greatly improved and validated as very effective during the flooding years of 2007 through 2011.

4.7.10 Economic Summary

Project benefits (Table 4-16) are the reduction in projected future damages, which would result from project implementation. The probabilistic values of equivalent annual damage (EAD) and EAD reduced show the impact of uncertainty in evaluation of project benefits. The damages reduced (*i.e.*, project benefits) are shown in terms of annualized equivalent values as computed in the HEC-FDA program.

Table 4-16: Recommended Plan Economic Benefits

	Equivalent Annual Damages			Probability EAD Reduced		
Plan	**Without Plan**	**With Plan**	**Damages Reduced**	**0.75**	**0.50**	**0.25**
Armourdale						
KR3 Plan	$55,392.04	$ 3,935.00	$51,457.05	$36,287.34	$49,899.39	$63,998.28
Central Industrial District						
KR3 Plan	$ 8,867.90	$ 3,638.32	$ 5,229.58	$ 1,583.39	$ 3,769.20	$ 7,442.57
Total	**$64,259.94**	**$7,573.32**	**$56,686.63**	**$37,870.73**	**$53,668.59**	**$71,440.85**

Note: October 2013 price level; 3.5% discount rate; 50-year period of analysis; $1,000's

Estimated project construction costs and OMRR&R costs were developed using the MII cost estimating system. These costs, along with annualized costs, annualized benefits, net economic benefits and the benefit-to-cost ratios are shown in Table 4-17: Recommended Plan Economic Summary. These values are based on October 2013 price levels, an interest rate of 3.5 percent, 50-year period of analysis and a 10-year construction period.

In the Kansas Citys study area, some major production facilities are either a sole producer of a specific product or are one of just a very few in the nation that produces that product. Proctor and Gamble is a prime example in the Armourdale Unit. Loss of production capability in these instances could be an economic loss to the nation unless consumers were able to find a similar product and made the choice to purchase the substitute product. However, these potential NED losses were not quantified for purposes of this study.

Induced damages would occur only if the Argentine Unit was raised and the downstream units, Armourdale and CID, were not. The Recommended Plan for raising the Armourdale and CID Units eliminates these induced damages. The Recommended Plan causes no new induced damages on other areas.

Table 4-17: Recommended Plan Economic Summary

Item	Recommended Plan
Interest Rate	3.5%
Construction period, years	10
Period of Analysis, years	50
Project First Cost	$313,402,000
Interest During Construction	$70,877,000
Investment Cost	$384,279,000
Annual Cost	
Amortized Cost	$16,383,200
OMRR&R	$336,500
Total Annual Cost	$16,719,700
Annual Benefits	$56,686,600
Benefit to Cost Ratio	3.4
Net Benefits	$39,966,900

4.7.11 Sensitivity of the Recommended Plan to Future Conditions

Both the future with and without condition scenarios are evaluated over a 50 year period of analysis to allow a consistent and appropriate comparison of alternatives. The period of analysis is the time horizon for which project benefits and project operation, maintenance, repair, rehabilitation and replacement (OMRR&R) costs are evaluated. The period of analysis begins with the base year condition (considering resources in the study area and economic and engineering factors) thought to exist in the first year a project alternative is expected to become operational. Extensive economic surveys of the whole Kansas Citys Levees study area were completed in 2002. Economic data developed for this analysis includes values, elevations and depth-damage relationships for homes, businesses, public facilities, roads, and railroads in the study area. Furthermore, a follow up survey was conducted in early FY2012 to update the economic field data. Engineering and economic data is also developed (projected) for a future year about 20 to 30 years out from the base year. The analysis years used in this Final feasibility study are 2026 for the base year and 2049 for the future year, with the total 50 year period of analysis ending in 2076.

In this study, certain assumptions related to the period of analysis were made:

- River stage uncertainty values were increased from 1.5 ft. to 1.8 ft. in the future year 2049; this reflects the increased difficulty in predicting stages far in the future.
- No significant increase in economic development is projected for the 50 year period of analysis as much of the protected area is essentially built-out.
- Beyond the future condition year of 2049, the expected annual damage is assumed to be constant in the remaining years of the period of analysis.

These assumptions provide a future without project scenario in which there are no substantial growth assumptions, which would influence project benefits.

4.7.12 Environmental Compliance

No significant environmental impacts have been detected to date. See Table 4-18 for the environmental compliance status.

Table 4-18: Environmental Compliance

Federal Law	
Archaeological and Historic Preservation Act, as amended, 16 U.S.C. 469, et seq.	Full
Clean Air Act of 1977, as amended, 42 U.S.C. 7609, et seq.	Full
Clean Water Act, as amended, (Federal Water Pollution Control Act), 33 U.S.C. 1251, et seq.	Full*
Coastal Zone Management Act, 16 U.S.C. 1451, et seq.	N/A
Endangered Species Act, 16 U.S.C. 1531, et seq.	Full
Estuary Protection Act, 16 U.S.C. 1221, et seq.	N/A
Federal Water Project Recreation Act, 16 U.S.C. 460-12, et seq.	Full
Fish and Wildlife Coordination Act, 16 U.S.C. 661, et seq.	Full
Land and Water Conservation Fund Act, 16 U.S.C. 460/ -460/-11, et seq.	N/A
Marine Protection, Research and Sanctuary Act, 33 U.S.C. 1401, et seq.	N/A
National Environmental Policy Act, 42 U.S.C. 4321, et seq.	Full

Table 4-18: Environmental Compliance

National Historic Preservation Act, 16 U.S.C. 470a, et seq.	Full
Rivers and Harbor Act, 33 U.S.C. 401, et seq.	N/A
Watershed Protection and Flood Prevention Act, 16 U.S.C. 1001, et seq.	N/A
Wild and Scenic Rivers Act, 16 U.S.C. 1271, et seq.	Full
Executive Orders, Memorandums, etc.	
Executive Order 11988, Floodplain Management, May 24, 1977 (42 CFR 26951; May 25, 1977)	Full
Executive Order 11990, Protection of Wetlands, May 24, 1977 (42 CFR 26961; May 25, 1977)	Full
Council on Environmental Quality Memorandum of August 11, 1980: Analysis of Impacts on Prime or Unique Agricultural Lands in Implementing the National Environmental Policy Act.	Full
Executive Order 12114, Environmental Effects Abroad of Major Federal Actions.	N/A
Executive Order 12898, Federal Actions to Address Environmental Justice in Minority Populations and Low-Income Populations, February 11, 1994.	Full
State and Local Policies	
Missouri Water Quality Standards	Full*

The compliance categories used in this table were assigned based on the following definitions:
Full Compliance (Full): Has met all requirements of the statute, Environmental Order (EO) or other environmental requirements for the current stage of planning.
Ongoing: Coordination ongoing, and should be completed prior to signature of FONSI.
Not Applicable (N/A): No statute, EO or other environmental requirement for the current stage of planning.
Full*: All necessary permits/certifications will be acquired prior to project implementation and/or construction.

4.7.13 System of Accounts Evaluation

The Principles and Guidelines for Water and Related Land Resources Implementation Studies (P&G) establish a system of four accounts for evaluation of alternative plans. The first of these accounts, National Economic Development (NED), evaluates the changes in the economic value of the national output of goods and services and is measured by the economic benefit, or reduced damages, resulting from the alternative plan, discussed previously. The remaining three accounts are:

- Environmental Quality (EQ). The non-monetary effects on significant natural and cultural resources.
- Regional Economic Development (RED). Changes in the distribution of regional economic activity that result from the plan.
- Other Social Effects (OSE). Plan effects from perspectives relevant to the planning process that are reflected in the other accounts.

An evaluation of the Recommended Plan for all four accounts is presented in Table 4-19.

Table 4-19 Evaluation of P&G System of Accounts

National Economic Development (NED)

	Armourdale		Central Industrial District	
	No Action	**Recommended Plan**	**No Action**	**Recommended Plan**
Project Cost	NA	$232,650,000	NA	$80,752,000
Annual Cost	NA	$12,343,800	NA	$4,375,900
Annual Benefits	NA	$51,457,100	NA	$5,229,600
Annual Net Benefits	NA	$39,113,200	NA	$853,700
BCR	NA	4.2	NA	1.2

Environmental Quality (EQ)

	Armourdale		Central Industrial District	
	No Action	**Recommended Plan**	**No Action**	**Recommended Plan**
Flooding	Expected Annual Flood Damage of $55.3 million.	Expected Annual Damage reduced by $51.5 million.	Expected Annual Flood Damage of $8.9 million.	Expected Annual Damage reduced by $5.2 million.
Air Quality	No immediate impact. Possible adverse future impacts	Temporary impacts during construction	No immediate impact	Temporary impacts during construction
Water Quality	No immediate impact. Possible adverse future impacts	Essentially no impact	No immediate impact. Possible adverse future impacts	Essentially no impact
Erosion and Sedimentation	No immediate impact. Possible adverse future impacts	Essentially no impact	No immediate impact. Possible adverse future impacts	Essentially no impact
Water Quantity	No impact	No impact	No impact	No impact
Ground Water	No immediate impact. Possible adverse future impacts	Essentially no impact	No immediate impact. Possible adverse future impacts	Essentially no impact
Aquifers	No impact	No impact	No immediate impact	No impact
Aquatic Habitat	No immediate impact. Possible adverse future impacts	Essentially no impact	No immediate impact. Possible adverse future impacts	Essentially no impact
Riparian Habitat	No immediate impact. Possible adverse future impacts	No impact	No immediate impact. Possible adverse future impacts	No impact
Upland Habitat	No immediate impact. Possible adverse future impacts	No impact	No immediate impact. Possible adverse future impacts	No impact
Floodplains (E.O. 11988)	No impact	No expected impact	No impact	No expected impact
Cultural Resources	No immediate impact	No impact	No immediate impact	No impact
Prime and Unique Farmland	No immediate impact. Possible adverse future impacts	No impact	No immediate impact	No impact
Economic Resources	Continued potential for property damage and business losses due to damaging flood events.	Significant reduction in property damage and lost business.	Continued potential for property damage and business losses due to damaging flood events.	Significant reduction in property damage and lost business.

Table 4-19 (continued)

Other Social Effects (OSE)

	Armourdale		**Central Industrial District**	
	No Action	**Recommended Plan**	**No Action**	**Recommended Plan**
Health and Safety	High level of flood risk in entire region with associated stress and anxiety, risk to regional health care system, and impacts to emergency access during floods. High potential for loss of life during flood fights.	Project would significantly reduce risk to public health and safety.	High level of flood risk in entire region with associated stress and anxiety, risk to regional health care system, and impacts to emergency access during floods. High potential for loss of life during flood fights.	Project would significantly reduce risk to public health and safety.
Economic Vitality	Current economy is strong. If catastrophic flood occurs, economic impacts would be extensive and long-lasting.	Project would significantly benefit the regional economy.	Current economy is strong. If catastrophic flood occurs, economic impacts would be extensive and long-lasting.	Project would significantly benefit the regional economy.
Social Connectedness	High levels of instrumental social support will continue throughout the region.	Armourdale area would see less frequent disruptions dne to flood fights.	High levels of instrumental social support will continue throughout the region.	The Central Industrial District would see less frequent disruptions due to flood fights.
Identity	Strong Hispanic heritage	Project would not likely affect cultural and community identity significantly	Strong European heritage with growing minority population.	Project would not likely affect cultural and community identity significantly
Social Vulnerability and Resilience	Armourdale highly vulnerable to catastrophic flood damage. Resilience of community may be lower due to lack of temporary housing options. Low income residents more vulnerable to short-term impacts of floodfighting.	Project would significantly reduce the Armourdale area vulnerability to floods, allowing focus on other social needs.	The Central Industrial District is vulnerable to catastrophic flood damage. Resilience of community may be lower due to lack of temporary housing options. Low income residents more vulnerable to short-term impacts of floodfighting.	Project would significantly reduce the Central Industrial District vulnerability to floods allowing focus on other social needs.
Participation	Residents in the study area exhibit a normal rate of participation in civic activities like flood fights, elections, and public meetings	Project would have little to no effect on civic participation.	Residents in the study area exhibit a normal rate of participation in civic activities like flood fights, elections, and public meetings	Project would have little to no effect on civic participation.
Leisure and Recreation	Residents of the area are active. Recreational facilities would continue to be provided as currently planned.	Project would have little to no effect on recreational opportunities.	Residents of the area are active. Recreational facilities would continue to be provided as currently planned.	Project would have little to no effect on recreational opportunities.

Table 4-19 (continued)

Regional Economic Development (RED)			
Armourdale		**Ceutral Industrial District**	
No Action	**Recommended Plan**	**No Action**	**Recommended Plan**
Continued potential for property damage and business losses due to damaging flood events.	Reduced flooding would enhance stability in employment in the Unit with potential for additional permanent employment opportunities; project construction would provide minor, short-term increase in construction employment; temporary increase in sales tax revenues during construction; property values would remain stable or improve, thereby increasing the local tax base; reductions in income attributable to flood damages, wage losses, traffic disruption costs, floodfight emergency expenditures.	Continued potential for property damage and business losses due to damaging flood events	Reduced flooding would enhance stability in employment in the Unit with potential for additional permanent employment opportunities; project construction would provide minor, short-term increase in construction employment; temporary increase in sales tax revenues during construction; property values wonld remain stable or improve, thereby increasing the local tax base; reductions in income attributable to flood damages, wage losses, traffic disruption costs, floodfight emergency expenditures.

4.7.14 Environmental Operating Principles

Under the seven Environmental Operating Principles (EOPs), the Corps of Engineers is mandated to proactively seek and consider ways to improve and sustain the environment. An existing project in an urban area such as Kansas City, with permanent structural features dating back several decades, has inherent limitations to the inclusion of viable environmental improvements. During the feasibility study, various candidate environmental measures were reviewed in recognition of the EOPs. In addition, flood risk management engineering measures were developed in a manner which sought to preserve, improve and sustain the environment. After review of the options and consideration of the conditions in this project area, it was generally determined that the best way to comply with the EOPs for this project, would be preservation of the continuity and value of habitat along and adjacent to the Kansas River bank line areas within the metropolitan area. The Recommended Plan has minimal impacts on existing habitat and wetlands and serves to protect the environmental and community fabric that has developed behind the existing levee system.

It is important to note the other Corps of Engineers projects underway in the general area that have substantial environmental benefits. The Missouri River Fish and Wildlife Mitigation Program provides for a long-term major restoration of areas along the Missouri River. The Riverfront Ecosystem Restoration Section 1135 project in the Kansas City reach of the Missouri River (near river mile 365.7) provides numerous environmental benefits along levee and floodwall areas and is a part of a larger effort to restore habitat and increase recreational opportunities along the Kansas City Missouri riverfront area. The Blue River project in the eastern sections of Kansas City and Jackson County also provides for a number of important environmental benefits in an urban setting. The benefits from all these other projects include: a) improvement of aquatic habitat by measures to improve water quality, bottom diversity, aquatic species spawning and rearing habitat; b) wetland restoration and natural vegetation development to improve habitat function and diversity; and c) improving the hydraulic connection and habitat continuity between riverine habitat areas, tributaries, and the Missouri River.

4.7.15 USACE Campaign Plan

USACE Campaign Plan. The USACE Campaign Plan contains four goals: Support the Warfighter. Transform Civil Works, Reduce Disaster Risks, and Prepare for the Future. Project formulation and alternative development furthered three of these four goals

Transform Civil Works: This study effort employed the current strategies in place for delivering enduring and essential water resource solutions. Review processes incorporated in this study included District Quality Control (DQC), Agency Technical Review (ATR), and Independent External Peer Review (IEPR). The ATR was conducted by an interdisciplinary team across several Corps Districts and coordinated with both the Flood Risk Management Center of Expertise and the Cost Estimating Directory of Expertise. The IEPR was managed by an outside organization employing independent technical experts. Customer and stakeholder engagement was encouraged throughout the planning process.

Reduce Disaster Risks: The overall study and recommendations as presented in the Interim Feasibility Report and this Final Feasibility Report present an integrated analysis of seven levee units to ensure overall system reliability and performance. Risk and uncertainty based models and methods were employed to examine the existing system and identify reliability deficiencies. The study team provided early and often communication of risk assessments, finding, and recommendations with the project sponsors and stakeholders using currently accepted terminology and concepts. Alternatives were chosen to reduce the flood risk to existing infrastructure and investment, and improve future system reliability. The Recommended Plan considers interactions and dependencies between units and sponsors and provides a complete plan for a safe, reliable, and resilient flood risk management system that mitigates disaster impacts to local community and the Nation.

Prepare for Tomorrow: The study effort employed the best available technical expertise and experience, and project management and leadership, to establish a dedicated, competent, and capable team to produce a quality project recommendation. The lessons learned by the team in the execution of this study will contribute to sustaining a culture of collaboration and innovation for delivering future solutions.

4.8 Implementation Requirements

Implementation responsibilities refer to actions and financial arrangements of Federal and non-Federal interests directed toward implementation of the Recommended Plan.

4.8.1 Institutional Requirements

The overall project schedule for the areas of interest in the Final Feasibility Report analysis is based upon the assumption that a positive Chief of Engineers' Report will be forwarded through the Assistant Secretary of the Army for Civil Works and the Office of Management and Budget to Congress for inclusion in authorizing legislation. Funding is assumed available at the earliest practical opportunity for new PED starts. Lack of initial PED funding will shift the schedule out accordingly until such time as the funding is made available. Additional refinements to the project schedule will be made as authorization and program guidance is received.
The project schedule provides for almost immediate start of design remedies beginning in FY2016, followed by award of construction contracts for the remedies, pending authorization, in FY20 through FY30. Several factors have been considered when projecting the sequence of future work:

- Construction contracts for different features can be undertaken simultaneously for increased efficiency.
- Improvements to the reliability of existing features will be implemented prior to increasing the levee height.
- Federal and Non-Federal construction funding is available in the years required
- Real estate actions are completed on schedule.

The project schedule (Table 4-20) reflects the information currently available and the current departmental policies governing execution of projects. It does not reflect program and budgeting priorities inherent in either the formulation of a national civil works construction program or the perspective of higher review levels within the Executive Branch. Consequently, the proposed schedule may be modified before it is transmitted to higher authority for authorization and/or implementation funding.

In meeting the area's needs for flood risk management, the Federal Government will be responsible for providing the Federal share of project costs and for implementing the Recommended Plan. The Kansas City District will develop the Project Management Plan sections needed for guiding the PED (design) and construction of the project.
The non-Federal sponsors are fully aware of and able to comply with all non-Federal sponsor responsibilities as described within the Recommendation section of this report.

Table 4-20: Project Schedule

Date	Task
April 2014	Feasibility Report Approval by the Civil Works Review Board
July 2014	Approval of the Report of the Chief of Engineers recommending the project to Congress for authorization
October 2015	Execution of Project Design Agreement with Local Sponsor; initiation of Pre-Construction Engineering and Design Phase (pending availability of design funding)
October 2018	Execution of the Project Partnership Agreement with the Local Sponsor (pending construction authorization and availability of construction funding); Initiation of Land and Easement Acquisition by the Local Sponsor
October 2019	Initiation of Construction
October 2029	Completion of Project Construction

4.8.2 Fully Funded Cost Estimate

The fully funded cost estimate accounts for all costs through construction completion including inflation based on the current project schedule. The current and fully funded project costs are presented in Table 4-21.

Table 4-21: Cost Summary by Levee Unit – Recommended Plan

Levee Unit	Total	Federal (65%)	Sponsor (35%)	PED	LERRD	FRM
OCT 2013 PRICE LEVEL ESTIMATE						
Armourdale Unit	$232,650	$151,222.5	$81,427.5	$15,611	$4,463	$212,576
CID Unit						
Kansas Section	$80,177	$52,115.1	$28,062.0	$5,450	$2,591	$72,136
Missouri Section	$575	$373.8	$201.3	$41	$0	$534
Totals	**$313,402**	**$203,711.3**	**$109,690.7**	**$21,102**	**$7,054**	**$285,246**
FULLY FUNDED ESTIMATE						
Armourdale Unit	$296,660	$192,829	$103,831	$19,117	$5,253	$272,290
CID Unit						
Kansas Section	$102,000	$66,300	$35,700	$6,659	$2,945	$92,396
Missouri Section	$735	$478	$257	$51	$0	$684
Totals	**$399,395**	**$254,279**	**$145,116**	**$25,827**	**$8,198**	**$365,370**

Notes: All costs in $1,000's; Amounts include the estimated contingencies for each site; Totals in this table are rounded.

4.8.3 Cost Apportionment

This discussion of individual non-Federal sponsor amounts is based on the fully funded project costs in Table 4-16. Of the two non-Federal sponsors, the Kaw Valley Drainage District of Wyandotte County, Kansas (KVDD), is responsible for the largest non-Federal share. KVDD will be responsible for the non-Federal share of work on the Armourdale Unit and the Kansas Section of the Central Industrial District Unit. Total costs for these two project components are expected to total $312,827,000. The 35 percent non-Federal share is $109,489,500. KVDD will continue to be responsible for annual operation and maintenance costs including the approximately $336,500 in annual costs added by this project. In a letter to the Kansas City District Corps of Engineers dated April 12, 2006, KVDD asserted their capability and intent to fund non-Federal costs for design, construction, operation and maintenance functions related to these two project components. KVDD support for the recommended plan of this report was expressed by letter dated March 4, 2014. KVDD currently funds their operations through a tax levy on properties within their district and they have authority under state statutes to issue general obligation bonds to raise funds. Additional possible funding alternatives include increases to their tax levy, expansion of their statutory bonding authority, and identification of local funding partners. It is expected that the proposed projects will be implemented in phases spread over a number of years to lessen the annual impact to KVDD's annual budgets and operations.

The City of Kansas City, Missouri, is responsible for non-Federal cost sharing of the recommended plan in the Missouri Section of the Central Industrial District Unit, with an estimated total cost of $575,000. The non-Federal cost share responsibility would be $201,300. The City will continue to provide annual budgets for levee operations and maintenance in accordance with current practice. This project is not expected to create additional operations and maintenance costs. The City expressed its intent and capability to provide the required non-Federal share in a letter to the Kansas City District dated June 16, 2006. The City expressed support for the recommended plan of this report by letter dated February 27, 2014. The City anticipates providing the non-Federal share from the Public Improvements Advisory Committee (PIAC) Capital Improvement Funds; the same as they are currently funding such projects.
The Kansas City District is of the opinion, based on the current financial standing and past performance of these sponsors, that their financing plans are reasonable and that they will be capable of meeting their financial obligations for implementation of the Recommended Plan.

4.8.4 Updated Survey Datum Requirements

Current Corps of Engineers policy stipulates that the vertical survey datum of all projects must be based on the North American Vertical Datum of 1988 (NAVD 88). This feasibility study analysis relied heavily on existing information including original levee record drawings and prior reports, many of which were based on the National Geodetic Vertical Datum of 1929 (NGVD 29). Recent design and construction projects undertaken by the Kansas City District, including implementation resulting from the Interim Feasibility Report, have shown a difference of between two and four inches in the vertical elevations when converting from NGVD 29 to NAVD 88. This was determined not to be a significant difference for the purposes of this feasibility report analysis and recommendations. Future design phase efforts for implementation of the Recommended Plan will require updated survey data using NAVD 88. For additional detail and discussion of survey information refer to the Survey and Mapping Chapter to the Engineering Appendix.

4.8.5 Permits

Requirements for Section 404 of the Clean Water Act of 1972, as amended, will be met prior to any construction activity, as well as any permit requirements of the Missouri Department of Natural Resources and/or the Kansas Department of Health and the Environment for any construction activity near the stream channel. The completed 404 (b) (1) guidelines form is included in Appendix J of the 2006 EIS.

4.8.6 Views of the Non-Federal Sponsors
The non-Federal sponsors strongly support the Recommended Plan. On a daily basis, each of the sponsors accomplish the numerous actions necessary for keeping the project in good condition as evidenced by recent annual inspection reports and by the evaluations undertaken in the feasibility study. The sponsors will continue to provide full cooperation and are prepared to meet the necessary financial obligations associated with the Recommended Plan.

The sponsors are fully aware of and in agreement with the requirements of the model Project Partnership Agreement. Both sponsors have previous experience on similar projects with Kansas City District that have utilized the model agreement with no requests for special conditions. It is anticipated that no special requirements will be requested or required for implementation of the Recommended Plan.

5 Environmental Considerations

5.1 Review of Previous Documentation and Current Conditions

The FEIS published in August 2006 (USACE, 2006b) included discussion of tentative alternatives, including environmental and cultural conditions and impact assessments, for the levee units and study areas discussed in this Final Feasibility Report. In accordance with the Council on Environmental Quality regulations for implementing NEPA, 40 CFR Part 1502.9(c)(1), Federal agencies "shall prepare supplements to either draft or final environmental impact statements if: (i) the agency makes substantial changes in the proposed action that are relevant to environmental concerns; or, (ii) There are significant new circumstances or information relevant to environmental concerns or bearing on the proposed action or its impacts." Due to the Armourdale and CID levee units schedule for construction initiation in 2018, and a seven year timeframe since completion of the 2006 FEIS, NEPA compliance review for these units was conducted to document any change in the scope of work and/or impacts to resources that occur within these levee units, and any potential changes to existing resources that may have occurred since the 2006 FEIS was finalized.

The Recommended Plan presented herein is within the project alternatives and geographic areas previously proposed and assessed; no substantial change has been made to the proposed action. Review of the project areas has shown no changes in the environmental conditions of the project area since publication of the FEIS, nor have there been changes in status, standards, or other factors that would affect the conclusions of the FEIS. Based on this review a new or supplemental EIS has not been prepared for the recommendations of this report. The FEIS is incorporated by reference and is currently available on the Kansas City District website, http://www.nwk.usace.army.mil/Missions/CivilWorks/ CivilWorksProgramsandProjects/KansasCitys,FloodRiskManagement.aspx. Federal and State resource agencies with jurisdiction over environmental resources reviewed this report and its findings during the Public Review period. No significant comments were received, as detailed in Appendix G.

The Armourdale and CID levee units are located within highly industrialized areas. As a result of industrialization, the resources within and adjacent to these levee units are severely disturbed. Impacts to resources due to the implementation of the recommended plan are documented in Section 4 of the FEIS. No significant impacts were documented to occur. Impacts primarily include a long-term, adverse visual impact due to the recommended landside levee raise, and minor, short-term, adverse construction related impacts. A review of current conditions and potential impacts of the Recommended Plan are presented in the following sections.

5.1.1 Affected Environment

Feasibility analysis and field reconnaissance was conducted for the 2006 FEIS to document the affected environment within each respective levee unit including water resources and water quality, geology and minerals, air quality, noise, visual quality, soils and prime farmland, hazardous, toxic and radioactive waste, cultural resources, floodplain terrestrial habitat, wetlands, fisheries, wildlife, threatened and endangered species, and the socioeconomic environment including recreation and environmental justice. The affected environment is addressed within section 3 of the FEIS. The FEIS addressed the impacts of the no action alternative, action alternatives that were not selected for implementation, as well as the recommended plan on the affected environment within each of the seven levee units. Impacts to resources are addressed in section 4 of the FEIS.

Throughout the study effort, Corps staff has maintained contact with the project sponsors, the Kaw Valley Drainage District and the City of Kansas City, MO, regarding ongoing property management and access to the levee units. No change in the property, or maintenance of the property, has been reported by the levee unit sponsors.

In July, 2013, Corps staff conducted field reconnaissance within the Armourdale and CID levee units, and the Water District Number One of Johnson County, Kansas (WaterOne) property that was identified in the FEIS as the proposed borrow location. Field reconnaissance was conducted by both driving and walking through the borrow area and levee units.

5.1.1.1 *Borrow Area*

WaterOne employees provided borrow area access and confirmed that WaterOne still owned the same property including the proposed borrow area addressed in the FEIS. WaterOne employees stayed on site and observed while CENWK conducted field reconnaissance of the WaterOne property.

The land use and land cover of the proposed borrow area has not changed compared to the land use and land cover observed and documented in 2006. The farmed wetland and associated vegetation, tree cover and crop field still exist in the same locations. The crop field was observed to be planted in soybeans. WaterOne employees reiterated that WaterOne has future plans to excavate monofills for lime storage. No new monofills were observed excavated since

2006. WaterOne employees were unsure when the next monofill would be excavated, but felt that the location would be adjacent to existing excavated monofills. No new borrow areas are proposed for use in addition to WaterOne property.

5.1.1.2 ***Armourdale Levee Unit***

The Armourdale levee unit was observed by driving the entire length of the levee and periodically walking through vegetated areas. The land use and land cover of the Armourdale Levee Unit has not changed compared to the land use and land cover observed and documented in 2006. The Armourdale unit still primarily consists of earthen levee and floodwall.

5.1.1.3 ***CID Levee Unit***

The CID Levee Unit was observed by driving along the entire length of the levee and walking through railroad property and fields. The CID unit still primarily consists of earthen levee and floodwall. No changes in land cover were observed. A minor change in land use was observed compared to the land use observed and documented in 2006. A large warehouse in the rail yard area at the upstream end of CID has been torn down and removed. The warehouse was observed in 2006 as being abandoned and in poor condition.

5.1.2 Threatened and Endangered Species

State and Federal listed threatened and endangered species for Wyandotte County, Kansas and Jackson County, Missouri were reviewed for changes in status since the 2006 FEIS. Additions to the Wyandotte County, KS list include the Federally endangered shoal chub, which inhabits the Kansas River. No work will be conducted within the Kansas River. Therefore, this species will not be impacted by the proposed project.

Additions to the Jackson County, MO list include the following terrestrial species: American badger, bald eagle, eastern collared lizard, Franklin's ground squirrel, thirteen lined ground squirrel, and tufted loosestrife. These species require specific habitats that are not available within the proposed project areas as the project areas are urbanized, industrialized, and severely disturbed. Riparian vegetation is available for the bald eagle. However, this species is not known to inhabit any riparian areas within the vicinity of the project area. No trees suitable for bald eagle roosting or nesting will be impacted by the proposed project.

Aquatic species include the lake sturgeon, longtail tadpole shrimp, plains minnow, sturgeon chub, and western silvery minnow. No work will be conducted within the Missouri River. Therefore, these species will not be impacted by the proposed project. One listed semi-aquatic species includes the northern crawfish frog. There is no northern crawfish frog habitat within the vicinity of the proposed work. Therefore, this species will not be impacted by the proposed project.

In response to the Final Feasibility Report (FFR) 30-day public comment period, neither the U.S. Fish and Wildlife Service, the Kansas Department of Wildlife and Parks, nor the Missouri Department of Conservation had comments concerning threatened and/or endangered species or their respective habitats. (See Appendix G.)

5.1.3 Hazardous, Toxic, and Radioactive Waste

The August 2006 Review of Completed Project, Kansas City Levees, Missouri and Kansas Interim Feasibility Report stated that "*The Interim Feasibility Report examines (and makes recommendations regarding) five of the seven levee units (Argentine, Fairfax-Jersey Creek, East Bottoms, North Kansas City, Birmingham). The Final Feasibility Report will address the remaining two units (Armourdale and CID). In accordance with 40 CFR 1500, the EIS addresses all seven levee units using projections of the tentatively preferred alternatives in the Armourdale and CID Units where firm detailed conclusions are not yet available. A supplement to the EIS will be developed to support the Final Feasibility Report.*" The Interim Feasibility Report recognized that there could potentially be a need for additional NEPA reporting, such as a Supplemental EIS (SEIS) if new information arose during the HTRW, geotechnical, and structural analyses to confirm the tentatively preferred alternatives for the Armourdale and CID Units and complete the feasibility study. The need for additional HTRW for Armourdale and CID was reiterated by the U.S. Environmental Protection Agency (USEPA) in the Lack of Objections rating for the 2006 DEIS (July 17, 2006). The USEPA as a cooperating agency for the study provided air quality, environmental justice, and HTRW information. Additionally, early in the feasibility study records and files were obtained from, and personal interviews conducted with, the Kansas Department of Health and Environment and the Missouri Department of Natural Resources. The USEPA DEIS review recognized that both Armourdale and the CID needed additional HTRW investigation. Table 2-1 of the EIS lists the tentatively preferred alternatives for Armourdale and CID as the nominal 0.2% event +3 ft. levee raise (KR3 plan) with underseepage controls. Sections 4.8.7 and 4.8.8 of these documents state that additional hazardous waste investigations for these units are needed and the FEIS states that the results of additional HTRW investigations will be used in selecting the preferred alternative for Armourdale and CID.

Despite the recognized potential, the tentative alternatives remained the selected alternative through the remaining analysis and no new substantive information arose. Additional HTRW investigation for Armourdale and CID levee units was completed in 2007 and is documented within Appendix D. A summary of the HTRW sites is provided below.

5.1.3.1 *Armourdale Levee Unit*

HTRW sites addressed within Appendix D of this FFR within the Armourdale levee unit that were not addressed within the DEIS or FEIS, as the analyses had not been fully conducted, include: Inland Container Corporation, Kaw Power Station, KC Hardwood Corporation (a.k.a. American Walnut), Kansas City Railcar Services, Auto Salvage Yard (formerly A to Z Production Plating), Sambol Packing Company, SELCO (formerly Chromium, Inc.), APAC –

Wilkerson and Union Pacific Railroad. The results of the additional HTRW documentation, annotated by section within Appendix D, include:

4.1.2 Inland Container Corporation: "*Based on information evaluated, no impacts to levee improvements resulting from HTRW concerns were identified. No further investigation is necessary.*"

4.1.3 Kaw Power Station: "*There is no longer evidence of contamination associated with this property. The property would only be encroached on if there was a need for a temporary easement. Therefore, no further investigation is needed.*"

4.1.4 KC Hardwood Corporation (a.k.a. American Walnut): "*There is no longer evidence of contamination associated with this property. The property would only be encroached on if there was a need for a temporary easement. Therefore, no further investigation is needed.*"

4.1.6 Kansas City Railcar Services: "*Due to the past use as a salvage yard, the property which falls into the limits of disturbance for the selected alternative should be more fully investigated during the Design Stage to ensure that the surface and subsurface soil are not contaminated and to determine how to dispose of any contaminated soil.*"

4.1.7 Auto Salvage Yard, Formerly A to Z Production Plating: "*Due to past use as a salvage yard, the property which falls into the limits of disturbance for the selected alternative should be more fully investigated during the Design Stage to ensure that the surface and subsurface soil are not contaminated and to determine how to dispose of any contaminated soil.*"

4.1.10 Schock Truck & Leasing: "*During the site visit, two AST's were seen from the levee road. The tanks were in good condition. A pile of debris lies between the tanks and jersey barriers that denote the edge of the property. It appears to be random construction debris. No other soil or groundwater contamination appears to be present at the site; therefore no further investigation is planned.*" The "*site visit*" as described in FFR Appendix D does not include the date that a "*site visit*" was conducted. The "*site visit*" is the last bullet in Section 3.0 HTRW SITE ASSESSMENT and is bulleted as "*Performed a site visit to the Armourdale Levee Unit.*"

4.1.11 Sambol Packing Co.: "*According to KDHE, Sambol was listed as having two UST's removed after the previous investigations were published. These were removed in 2000. KDHE found no evidence of contamination and considered the site closed. During the site visit, no contamination issues were seen. No HTRW concerns exist for this site; therefore, no further investigation is required.*"

4.1.12 SELCO (formerly Chromium, Inc.): "*The aerial photographs show the site was industrialize*" (the author did not state "industrialized") "*before 1951, as KDHE records had said. The current building for SELCO was in existence by 1983. The site visit did* *not indicate any current hazardous waste issues occurring at the site. As only very low concentrations of metals and VOCs below KDHE action levels at the site, no further investigation is required at this location. The previous buried construction debris, if excavated, may be taken to a solid waste landfill."*

4.1.14 APAC – Wilkerson: "*During the site visit, APAC was still using the properties as a storage yard for their construction equipment. No contamination is known to be present at this site. Based on information evaluated, no impacts to levee improvements resulting from HTRW concerns were identified. Therefore, no investigation is necessary.*"

4.1.15 Union Pacific Railroad: "*During a review of EPA documents, a permit for discharging wastewater and sludges was requested by UPRR. A 1982 document states that test results of the wastewater classified it as non-hazardous. The UPRR withdrew its permit application. While a potential exists for previous contamination resulting from spills along the railroad lines, all known contamination has been remediated from various UPRR sites. Therefore, no further investigation is warranted for the railroad yards.*"

5.1.3.2 ***CID Levee Unit***

One potential HTRW site is addressed by Appendix D of this FFR within the CID Levee Unit: River View Properties Inc. The summary of the results, annotated by section within Appendix D, states:

4.2.1 River View Properties Inc.: "*Potential HTRW concerns have been identified within the study area between station 40+31 and 51+00. Potential encroachment into this area associated with the levee raise is proposed to be avoided by steepening the landside levee slope rather than extending the landside toe. No other locations of HTRW concerns have been identified at this time.*"

The HTRW investigation conducted to complete the HTRW analysis for this FFR revealed no new substantive information that changes the conclusions of the original EIS or warrants additional NEPA. HTRW investigation has been conducted to the practicable extent within both the Armourdale and CID unit.

5.1.3.3 ***Remaining Areas of Concern***

Considering the urban industrial nature of both areas, it is recognized that there is a residual risk that unidentified concerns are present and HTRW may be encountered during project implementation. Additional soil sampling and testing will be conducted as part of the design phase to verify the limits of known contamination, as well as close monitoring of material excavated during the project construction to ensure that any previously unknown HTRW uncovered is properly handled and disposed. Identified concerns and proposed actions at each of the known HTRW locations are listed in Table 5-1. Any and all removal of contaminated soils or other contaminated materials will be 100% local sponsor responsibility (including cost). All removal of known contaminated soils or other materials must be completed prior to construction.

Table 5-1: Armourdale Unit Areas of HTRW Concern

Location	Proposed Action
43+00 to 63+00 Proctor & Gamble Manufacturing	Levee raise methods proposed in this reach include T-wall on the existing levee, levee replacement with new floodwall, and floodwall replacement with floodwall, all of which avoid expansion of the levee toe into the area of concern. A slurry cutoff wall will be installed to avoid discharge of contaminated groundwater that may occur with relief wells. Any construction debris encountered near the former Fire Training Area should be removed, sampled, and properly disposed.
110+00 to 130+00 Auto Salvage Yards KC Railcar Services	Levee raise methods proposed in this reach include T-wall on the existing levee and a landside levee raise, which would encroach upon the area of concern. The property will be more fully investigated during the design phase to ensure that surface and subsurface soils are not contaminated and to determine how to dispose of any contaminated soils.
130+00 to 157+00 Trimodal	The levee raise method proposed in this reach is a T-wall on the existing levee. Intrusive activity is limited to areas outside the area of concern. Construction of any haul roads outside the existing right-of-way must be coordinated the Kansas Department of Health and Environment.
278+00 to 293+00 PBI Gordon Corporation	The levee raise method proposed in this reach is a modification of the existing floodwall. No HTRW concern is expected as no invasive activity is planned within the area.

5.1.4 Wetland Delineation and Potential Impact Assessment

In accordance with the statements made in sections 4.11.7 and 4.11.8 of the FEIS, wetland delineation and impact assessment was conducted for the Armourdale and CID levee units to complete the feasibility study. Review of NWI mapped wetlands revealed two NWI-mapped wetlands within the CID unit and seven NWI-mapped wetlands within the Armourdale unit. Wetland delineation was conducted by walking through all of the NWI-mapped features and adjacent land.

Only mapped feature #4 within the Armourdale unit was determined to be a wetland based on the presence of wetland vegetation and wetland hydrology. No soil samples were needed as this feature consists of open water with a wetland fringe. All other areas were riparian forest/riparian scrub-shrub or old field with 10YR3/2 silty clay, silty clay loam, no mottles. Descriptions of enumerated features and associated work include:

> Area 1 is a NWI-mapped feature that does not exist. The area west of the bridge was dominated by cottonwoods (*Populus deltoides*). No depressions or potential wetlands were observed. Work proposed in this area includes replacing existing levee with a floodwall (station 58+00 to 60+40), which would impact the existing levee. No work will occur riverside of the levee.

Area 2 was dominated by riparian vegetation including cottonwoods, box elder (*Acer negundo*), and silver maple (*Acer saccharinum*). Occasional grapevine (*Vitis riparia*) was also observed. The topography of this area is very irregular with depressions. No primary or secondary wetland hydrology indicators were observed. Work proposed includes installing a T-wall on the existing levee. No impacts to this area would occur due to construction as no work is proposed riverside of the levee and levee work would occur at a distance of approximately 230 ft.

Area 3 was very similar to Area 2 as the topography was highly variable with depressions and the landscape was dominated by scrub-shrub box elder with a silver maple fringe. Occasional grapevine was also observed. No direct hydraulic connection with the Kansas River was observed. No primary or secondary wetland hydrology indicators were observed. Work proposed includes installing a T-wall on the existing levee. No impacts to this area would occur due to construction as no work is proposed riverside of the levee and levee work would occur at a distance of approximately 230 ft.

Area 4 consisted of an open water feature (pond) with an emergent wetland fringe dominated by barnyard grass (*Echinochloa crusgalli*) and millet (*Panicum dichotomiflorum*) with some areas of vegetation dominated by reed canarygrass (*Phalaris arundinacea*). Forested wetland fringe comprised of silver maple and box elder was observed on the western side of the pond. The area of wetland fringe varied, but measured a maximum of approximately 30' in width. Hydrology to this area is provided by a 42" inch storm sewer. No impacts to this feature would occur due to construction as the work proposed is a landside levee raise.

Area 5 consisted of old field vegetation including vetch, goldenrod, and fescue with a fringe of eastern red cedar. No impacts to this feature would occur due to construction as the work proposed is a landside levee raise.

Areas 6 and 7 are additional NWI-mapped features located to the west. These features are storm sewers that drain into the Kansas River. Riparian vegetation including box elder and cottonwood dominates within the vicinity of these drainages. No wetlands were observed within the vicinity of these storm sewers. No impacts would occur within the area of these features as proposed work consists of replacing the existing levee with floodwall.

NWI-mapped wetlands within CID include two features located down gradient of the existing levee located between the James Street Bridge and I-70 Eastbound Bridge. One linear depression was observed about 40 ft riverward of the levee toe. The depression consisted of downed cottonwoods presumably cut down by the Kaw Valley Drainage District for maintenance purposes. This area receives direct precipitation and runoff from the adjacent levee slope. No soil samples could be taken within the depression due to multiple downed cottonwoods and no indicators of wetland hydrology could be observed. Soils are mapped as Eudora-Urban Land Complex. No impacts are anticipated to occur in this area as the proposed work consists of a landside levee raise.

The assessment of the proposed project in both units determined that these wetlands are not expected to be impacted by implementation of the Recommended Plan.

5.1.5 Incorporation of Previous USFWS Recommendations

Recommendations from the USFWS received during the 2006 review of the IFR were reviewed for their applicability to the recommended plan of this Final Report. The previous comments are enumerated below and are followed by the manner in which the Corps is addressing these recommendations in both Phases of the project:

1. Riparian and wetland habitats should be avoided to the maximum extent practicable when selecting borrow sites for the proposed levee raises and compensatory mitigation should be undertaken for unavoidable impacts.

    Impacts to natural resources including wetlands, islands, snags, riparian and upland trees were avoided during plan formulation and design and are being avoided during construction to the extent practicable. An example of avoidance is the removal of clearing the Argentine riparian foreshore from the alternatives for that levee unit. The Phase 1 project implementation will include compensatory mitigation for significant unavoidable resource impacts as needed. No impacts are expected from the Phase 2 recommendations.

2. Levees should be seeded with warm season grasses such as switch grass.

    Levee seeding is conducted in accordance with the information provided in the operation and maintenance section within *the "Guidance For The Design and Construction Within The Critical Area Of Constructed Flood Control Projects"* (http://www.nwk.usace.army.mil.localprotection/guidance.html), MAINTENANCE Chapter, paragraph 2.2 to 2.2.10. The seeding requirements meet 33 CFR 208.10 Part B section, Levee Maintenance. This requirement assures that levee slopes are mowed on a regular basis for close inspection of the slopes. Close inspection is required to detect settlement, sloughing, slope instability, erosion, the presence of burrowing animals, the presence of debris, encroachments that tend to weaken levees, rutting, depressions or other effects. Regular mowing also assures that deep-rooted vegetation will not become established on levee slopes. Levees will not be seeded with warm season grasses such as switch grass as warm season grasses are not amenable for use on levee slopes meeting the above requirements.

3. Removal of mature cottonwoods, and other native vegetation should be avoided where possible, and if they are removed, replace woody vegetation by establishing 2 acres of native vegetation for every acre impacted.

The removal of woody vegetation has been avoided to the extent practicable. There are trees within the area of construction area of the Phase 1 Fairfax-Jersey Creek sheetpile wall site that will be removed and mitigated at a 2:1 ratio. No other levee unit work, including the Phase 2 recommended plan, is anticipated to result in impacts to woody vegetation.

4. The Corps should create wetland mitigation habitat to compensate for the loss of wetland acreage from the construction of the project. Because an, as yet, unknown number of acres of farmed wetland may be directly impacted, it may be necessary to restore non-wetland habitat to wetland habitat. Farmed wetlands should be mitigated at a 1.0 to 1.0 ratio.

As evaluated in the 2006 EIS, a farmed wetland measuring 0.17 acres is located within the proposed borrow area property owned by Johnson County Water District #1 (WaterOne). This borrow area will provide the material needed for both Phase 1 and 2 levees raises. This farmed wetland will be mitigated at a 1.0 to 1.0 ratio if it is impacted by borrow activities It is unknown at this point in time if the farmed wetland will be impacted as the exact location of borrow within the site is currently unknown..The location of borrow will be at the location of WaterOne's next monofill excavation, which is currently unknown per personal communication with WaterOne on January 9, 2014.

5. Since channelization and levee construction have already resulted in dramatic loss of riparian and wetland habitats in the Missouri and Kansas River basins, the alternative to remove riparian vegetation to increase discharge capacity of the lower Kansas River should be dropped from further consideration.

The alternative to removed riparian vegetation to increase discharge capacity of the lower Kansas River was dropped from further consideration.

6. Encourage wetland development and hydrological re-connection to the river at existing borrow areas landward of the levee units.

Opportunities for environmental measures within the system are being considered in combination with potential mitigation requirements planned under the Phase 1 recommendations. The Phase 2 recommended plan does not include mitigation potential nor any environmental development measures.

7. Provide river access at the Argentine Levee segment.

Providing river access at the Argentine levee segment or the construction of an access road over the Argentine unit is not within the scope of work for this project.

8. Establish native vegetation (trees and shrubs) riverward of levee segments where riparian woodlands are sparse or nonexistent.

The feasibility of woody vegetation establishment is being considered in locations where impacts to woody vegetation occur due to construction, currently only expected at the Fairfax Jersey-Creek Unit Sheetpile Wall project site of Phase 1. Establishment of additional trees and shrubs in areas where no impact is expected to occur is not proposed.

9. Potential for aquatic and wetland restoration at Liberty Bend Cut-off just downstream of Kansas City should be explored.

No aquatic or wetland impacts are currently anticipated for the Phase 2 recommended plan. The Phase 1 recommendations do not anticipate a need for aquatic restoration and have proposed wetland restoration at the site of impact. If, during implementation, previously unforeseen impacts result requiring mitigation for fish and wildlife, the Liberty Bend cut-off site will be considered.

10. Consult with State wildlife agencies in regards to state-listed threatened and endangered species.

Coordination was conducted and completed with the Kansas Department of Wildlife and Parks and the Missouri Department of Conservation. Impacts to state-listed threatened and endangered species will be avoided to the extent practicable.

11. Conduct surveys for the presence of nesting birds in areas slated for clearing and grubbing.

A CENWK biologist is conducting surveys for the presence of nesting birds within all levee units prior to construction.

12. Best Management Practices should be included within the project specifications to avoid and minimize erosion and petrochemical spills within construction areas.

Erosion control measures are included in the project specifications where applicable and include silt fences, straw bales, and other suitable mechanisms. Measures being used to prevent the loss of petrochemicals into waters of the U.S. include the designation of staging areas for chemical storage away from streams, fueling heavy equipment away from streams, and the proper disposal of contractor generated waste. Contractors are also required to submit an environmental protection plan prior to initiating construction activities.

13. Invasive species have been identified as a major factor in the decline of native flora and fauna and their ecosystems. Nearly half of the species currently listed as Threatened

or Endangered under the U.S. Federal-Endangered Species Act are considered to be at risk primarily because of competition with and predation non-indigenous species (Nature Conservancy 19915; Wilcove et al. 1998). Human actions are the primary means of invasive species introductions. Prevention of introductions is the first and most cost-effective option for dealing with invasive species (Global Invasive Species Program Toolkit). Executive order 13112 Section2 (3) directs Federal agencies to not authorize, fund, or carry out actions that it believes are likely to cause or promote the introduction or spread of invasive species in the United States or elsewhere and to ensure that all feasible and prudent measures to minimize risk of harm will be taken in conjunction with the actions. Therefore, we recommend that the following Best Management Practice (BMP) be implemented during construction of the levees.

All equipment brought on site will be thoroughly washed to remove dirt, seeds and plant parts. Any equipment that has been in any body of water within the past 30 days will be thoroughly cleaned with hot water (hotter than 40° C or 104° F) and dried for a minimum of five days before being used at this project site. In addition, before transporting equipment from the project site all visible mud, plants, and fish/animals will be removed, all water will be eliminated, and the equipment will be thoroughly cleaned. Anything that came in contact with the water will be cleaned and dried following the above procedure.

> Levee unit construction is not anticipated to cause or promote the introduction or spread of invasive species in the United States or elsewhere. Best Management Practices are always included within project specifications and include a requirement for heavy equipment washing and drying prior to and following construction.

5.1.6 Cultural Resources

The cultural resource evaluation of the project area found no archaeological sites or historic structures listed on or eligible for listing on the National Register of Historic Places (NRHP). The project area, heavily disturbed by past levee and urban related construction, was found unlikely to contain previously unidentified archaeological sites eligible for inclusion in the NRHP. Cultural resource findings were coordinated with both the Kansas and Missouri State Historic Preservation Officers who concurred with Corps of Engineers recommendations for no further investigations unless an unanticipated discovery is encountered during construction. See also the 2006 FEIS (USACE, 2006b) section 4.2.7 Armourdale Levee Unit Raise Alternatives and section 4.2.8 CID Levee Unit Raise Alternatives.

5.1.7 Environmental Justice

The Executive Order on Environmental Justice (12898) requires consideration of social equity issues, particularly any potential disproportionate impacts to minority or low-income groups. This is to ensure that issues such as culture and dietary differences are taken into consideration to ensure that adequate risk is evaluated (USEPA, 2003). To determine potential impacts to minority or low-income groups, the racial and income composition of the individual census tracts within, and adjacent to the study area, were examined using 2000 census data. The focus of Executive Order 12898 provides for the protection of both minority and low-income groups. The results of the Environmental Justice evaluation show that a significant minority population (>25%) is present within the Armourdale and CID levee units. A significant number of persons living at below the national poverty level also reside within the Armourdale Unit. There exists a minor potential for the Recommended Plan to have limited impacts on the Armourdale and CID populations and community cohesion.

Implementation of a levee raise of the Argentine Unit as recommended and approved in the Interim Feasibility Report, prior to any raise in the Armourdale and CID Units, may induce flood damages on the downstream units under extremely rare flood events until such time as equal levels of protection are attained at all three levee units. These potential induced damages are considered temporary and would only occur in the event of a major flood (more rare then the nominal "250 year" event). Impacts to the Armourdale and CID populations are limited by the rarity of coincident circumstances which must occur in order to produce the induced damages. Because significant populations of low income families and cultural and racial minorities reside and work within all the Kansas River Units, there would be no significant difference between implementation of the one unit prior to another. The project would meet the intent of protection of minority and low income populations under Executive Order 12898.

5.1.8 Secondary and Cumulative Impacts

Potential cumulative impacts relating to past, present, and projects within the foreseeable future were evaluated along with the preferred plan to determine the level, if any, of impacts upon the physical and natural environment along the Kansas and Missouri Rivers. The Recommended Plan involves a combination of levee raises and appurtenances that lies primarily within the footprint of the existing levee system. As a result of project implementation, impacts to the existing river systems are relatively minor and not considered significant. Compared to past activities and current operations within these reaches of the rivers, the additional minor impacts created by the increased levels of protection do not create significant additional or cumulative impacts to the environment. Induced damages are discussed in section 3.6.4 Induced Damages.

Executive Order 11988 requires federal agencies to avoid to the extent possible the long and short-term adverse impacts associated with the occupancy and modification of flood plains and to avoid direct and indirect support of floodplain development wherever there is a practicable alternative. The guidelines address an eight-step process that agencies should carry out as part of their decision-making on projects that have potential impacts to or within the floodplain.

1. Determine if a proposed action is in the base floodplain (that area which has a one percent or greater chance of flooding in any given year).

Per FEMA mapping, the areas currently protected by the Kansas Citys Levees are outside of the 1% event floodplain. However, the existing levees themselves which are proposed to be modified are part of the 1% floodplain.

2. Conduct early public review, including public notice.

The Corps' Notice of Intent to prepare an EIS was published in the Federal Register on January 10, 2001. The initial scoping process was conducted during the summer/fall of 2003 and included meetings with local, state and Federal agencies, organizations and the general public. On August 20, 2003, the Corps held a public information/scoping meeting to present information on the study and to receive input from the public on resources in the affected area, alternatives and potential impacts.

A notice of availability for the Draft Environmental Impact Statement (DEIS) was published in the Federal Register on June 2, 2006. A public meeting was held in Kansas City, Kansas on July 13, 2006. Comments on the Draft Interim Feasibility Report and the DEIS were received during a 45-day comment period via email, the public meeting, and letters. The comment period ended July 17, 2006.

A notice of availability for the Draft Final Feasibility Report was mailed on November 22, 2013. Comments on the report were received during a 30-day comment period via email and letters. The comment period ended December 21, 2013.

3. Identify and evaluate practicable alternatives to locating in the base floodplain, including alterative sites outside of the floodplain.

The proposed project is for modification and improvement of an existing levee system and is generally limited to the current location and features of that system.

4. Identify impacts of the proposed action.

No floodplain impacts are expected from the proposed action. These protected areas are heavily urbanized and intense development has already been in place for many years. Significant development is not anticipated to be induced by the proposed levee project because very little open space remains and recent development has primarily consisted of improving old structures, or razing old structures and replacing with new structures.

5. If impacts cannot be avoided, develop measures to minimize the impacts and restore and preserve the floodplain, as appropriate.

No impacts are expected.

6. Reevaluate alternatives.

As stated in the response to Question 3, the proposed project is for modification and improvement of an existing levee system and is generally limited to the current location and features of that system.

7. Present the findings and a public explanation.

Study findings and recommendations as documented in this feasibility report were published for public review and comment in November 2013. Comments received are included in Appendix G.

8. Implement the action.

The proposed plan detailed in the report is recommended for approval and authorization.

5.1.9 Environmental Mitigation

After considering the environmental features of the project area, there is little potential for impacts to the existing environment from project area construction activities. The project area is primarily within urban industrial areas with little or no environmental features, or is contained with the existing project easements and rights-of-way which are already clear of trees in accordance with current O&M guidelines. Construction activities may include the clearing of grasses, weeds, and incidental immature plants and shrubs, but impacts to mature trees and wetlands are not anticipated. Therefore, mitigation funding has not been included within the Recommended Plan estimated costs.

5.1.10 Climate Change Considerations

Corps of Engineers guidance on climate change adaptation inputs for inland hydrology is at the draft final stage of production, and has not yet been officially released for use. As such, there was no guidance in place when the hydrologic analysis was conducted (finalized 2006) for the Kansas City Levees Feasibility Study. The proposed USACE guidance will initially recommend a qualitative approach. A summary of the qualitative approach as would be applied to the Kansas City Levees is provided below.

The climate of northeast Kansas trends toward a continental weather pattern of cold winters and hot, humid summers. The average temperature in 2013 at Topeka, KS (which represents the northeast portion of Kansas) was 60 degrees. The average high temperature was 73 and average low temperature was 47. The average yearly precipitation was about 37 inches of moisture.

A model of future conditions for the central plains of the United States was created by the National Oceanographic and Atmospheric Administration's (NOAA) National Environmental Satellite, Data and Information Service in a report issued in January 2013. This report is an assessment of Climate Trends and Scenarios into the next 50 to 100 years. The report cites that over the past period of record for the Kansas River basin, both temperature and precipitation has trended above normal, especially over the last 50 years. To account for climate change in the meteorological conditions, the future forecast of conditions in the region takes into consideration the past temperature and precipitation records, and then considers future modeled conditions in the area through 2070. According to the NESDIS report, a warming trend of about 3-5 degrees F and a precipitation trend very slightly toward wetter conditions can be expected through the next 50 years although significant uncertainty is expected with these estimates. Based on this slight trend toward wetter conditions frequency flows over the study basin may increase, but these increases are being treated in this evaluation to be retained within the bands of uncertainty in the Existing Condition Feasibility hydrologic analysis.

5.2 Determination of Need for Additional NEPA Documentation

The final geotechnical and structural evaluations were required to narrow the array of measures considered for implementing the projected alternative plans, and to determine the material quantities needed for cost estimating and subsequent economic analysis. The information gathered and analyzed confirmed the initially projected alternatives as feasible and cost-effective recommended plans for each unit. No changes to the initially projected project scope or location were made and no new substantive information was found that changed the affected environment or project impacts. With the inclusion of this information within the FFR, there is no need for additional NEPA compliance documentation, including an SEIS. The tentatively preferred alternatives for the Armourdale and CID units as described in the IFR, DEIS, and FEIS as the nominal 0.2% event-year+3 levee raise (KR3 plan) with underseepage controls is still the preferred alternatives for these units and all feasibility analysis is complete.

The record of decision (ROD) for the Kansas Citys Levees Project signed November 21, 2007 by the Assistant Secretary of the Army for Civil Works recognized that "*the public interest will best be served by implementing the improvements identified and described in the interim feasibility report and the Final Environmental Impact Statement*". The improvements as described in the FEIS is documented as levee raises and underseepage improvements for the Armourdale, CID, Argentine, East Bottoms, Fairfax-Jersey Creek, and North Kansas City Levee Units, and the no action alternative for the Birmingham Levee Unit. Therefore, although the ROD omitted to mention the Armourdale and CID units by name, these units are addressed within the FEIS and included within the aforementioned quote from the ROD.

5.3 Environmental Considerations Conclusion

The 2006 scope of work for both the Armourdale and the CID Levee Units, documented as "tentative" in the DEIS, FEIS, IFR, and confirmed as the preferred alternatives (KR3) with the completion of HTRW and engineering analyses as documented in this FFR, consists of landside levee raises with underseepage control improvements to provide equal levels of protection among the Kansas River Levee Units. The preferred alternatives, locations, and scope of work for these levee units have not changed. There was no observed change in the affected environment of these levee units during the July 15, 2013, field reconnaissance with the exception of the aforementioned removal of a large, degraded warehouse formerly located within the CID. Similarly, there were no substantial changes in environmental consequences associated with the project, including the results of additional HTRW, geotechnical, and structural analysis conducted for the FFR. The risk of residual HTRW is recognized as a project risk during construction. Therefore, the impacts to resources as a result of the implementation of the recommended plan within the Armourdale and the CID Levee Units remain unchanged as reported within the 2006 FEIS, with no additional short- or long-term adverse impacts identified or anticipated.

The IFR, DEIS and FEIS all recognized that the only additional information needed to complete feasibility was limited to minor HTRW investigation within the Armourdale and CID levee units. The DEIS, FEIS, IFR, and FFR all address the seven levee units that comprise the Kansas City Levees, including Armourdale and CID. All remaining HTRW investigation was completed and is documented in Appendix D. NEPA compliance is complete. There is no need for additional NEPA documentation.

Based on review of the FEIS and the evaluations in this FFR, environmental impacts of the Recommended Plan are limited within the project area. Environmental impacts to the project area are considered minor or not significant with many impacts temporary in nature during construction activities. Cultural resource assessment of the project area showed no significant archaeological sites or historic structures impacted by the Recommended Plan; thereby resulting in no significant impacts. However, if significant archaeological or cultural materials are discovered as the project progresses, then appropriate measures for coordination, documentation, and preservation, if needed, would be undertaken. No significant long term socio-economic impacts were identified for the populations within the project areas. Temporary impacts associated with construction activities would occur but are considered not significant. Based on the environmental analysis, implementation of the Recommended Plan would result in no significant impacts to the environment. As the formal decision document, a new ROD that states the alternatives considered and describes the selected alterative for the Armourdale and CID levee units will be prepared for review and approval by the Major Subordinate Command.

6 Public Involvement, Review, and Consultation

Review of the Environmental Impact Statement (EIS) supporting these project recommendations was conducted during the Phase 1 Feasibility Study in accordance with the National Environmental Policy Act (NEPA), and Corps of Engineers policies. The NEPA and EIS processes require full disclosure of present, future and cumulative, economic, environmental and social impacts that might occur as a result of implementing the preferred plan examined within this study. Following is a general description of the public involvement process applicable to the final feasibility study.

Public involvement provides for general public and Agency input and review within the overall NEPA process. The Corps actively solicited input from numerous Federal, State and local agencies, businesses, and organizations. Subsequent to Corps Headquarters (HQ-USACE) approval for public release of the draft Final Feasibility Report a Notice of Availability appeared in the Federal Register. Notice of the report availability was sent to the study sponsors, elected officials, tribal governments, Federal agencies, state, county, city and local governments, environmental groups, businesses, individuals, news media, libraries, and neighborhood groups and other individuals and organizations on the project mailing list. A press release was made and the project website updated to include the released information. The Draft Report was made available for public review on the website, at area public libraries and at the Kansas City District Corps of Engineers office. The comment period on the draft documents ran for 30 days after the Notice of Availability appeared in the Federal Register. All substantive comments received during this period are included in Appendix G with responses.

6.1 Public Scoping Meetings

Scoping meetings for the feasibility study and Environmental Impact Statement were held during Phase 1 of the Feasibility Study. Invitations and announcements for the scoping meetings were sent to the study sponsors, elected officials, tribal governments, Federal agencies, state, county, city and local governments, environmental groups, businesses, individuals, news media, libraries, and neighborhood groups.

Issues and concerns identified by Agencies regarded potential impacts to downstream areas resulting from implementing any flood risk management measures, economic development of the riverfront area, transportation impacts on bridges, highways, barge traffic, channelization of the Kansas and Missouri Rivers, the potential loss of natural resources, impacts on historic trails and sites, and opportunities for Missouri River recreation and levee trails related to the Metro Green Trail System.

The public recognized the need for effective flood risk management; however they also recognized other needs. The priority needs voiced by the public were related to Missouri River recreational opportunities. Many public comments related to incorporating walking and bicycling trails into the Kansas Citys levees system. Comments also related to the interest and need for parks along the rivers and/or levees. The public also voiced concern over the lack of public access to the Missouri River and Kansas Rivers due to the continuous linear nature of the levees. There were some questions concerning peak flows, scouring, and the water resource models that would be used when addressing urban flood risk management issues.

6.2 Views of Other Agencies

Extensive coordination with several State and Federal agencies took place during development and evaluation of the Recommended Plan. The following agencies were coordinated with and in some cases have provided comments or participated in the review of this project:

- Kansas Department of Health and Environment
- Kansas Department of Wildlife and Parks
- Kansas State Historic Preservation Office
- Missouri Department of Conservation
- Missouri State Historic Preservation Office

- US Environmental Protection Agency
- US Fish and Wildlife Service
- US National Parks Service

Agency views or concerns expressed during the scoping process or through ongoing study coordination, focused on:

- potential or actual contamination within the industrialized areas of the levee units,
- environmental justice for local communities during the formulation of alternatives,
- potential channelization of the Kansas and Missouri Rivers
- quality of the foreshore riparian habitat along the rivers,
- wetlands within the project area,
- threatened and endangered species,
- cultural resources or historic properties that may be encountered.

Agencies have provided concerns or comments through the public scoping process, through a Planning Aid Letter, through coordination and submittal of the draft Fish and Wildlife Coordination Act Report, through coordination letters to the State Historic Preservation Officer, and through day to day contact with appropriate agencies as the formulation process and EIS developed. As a cooperating agency, the EPA has provided specific input and review on contaminant issues, air quality information, and an Environmental Justice evaluation pursuant to Executive Order 12898.

6.3 Status of Corps of Engineers Review Process

6.3.1 Policy Compliance Review

The Alternative Formulation Briefing (AFB) to HQ-USACE occurred on April 24, 2013. The Project Guidance Memorandum (PGM) was submitted to HQ-USACE on July 23, 2013, detailing the AFB comments and issues and the Kansas City District plan for resolution prior to public review of the Draft Final Report. The Draft Feasibility Report was submitted to HQ-USACE in September 2013 for policy compliance review and backcheck of the responses and actions taken per the PGM. Additional comments on the DFR were issued by HQ-USACE in November 2013 and have been incorporated into this Final Feasibility Report.

6.3.2 Agency Technical Review

An Agency Technical Review (ATR) has been completed, led by the Louisville District and including reviewers from several other Corps District offices. The ATR was conducted as defined in the project's Review Plan to comply with the requirements of EC 1165-2-214. During the ATR, compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This included review of: assumptions, methods, procedures, and material used in analyses, alternatives evaluated, the appropriateness of data used and level obtained, and reasonableness of the results, including whether the product meets the customer's needs consistent with law and existing US Army Corps of Engineers policy. The ATR also assessed the District Quality Control (DQC) documentation and made the determination that the DQC activities employed appear to be appropriate and effective. All comments resulting from the ATR have been resolved and the comments have been closed in the Corps' DrChecks review tracking system.

Significant concerns and the explanation of the resolution are as follows: Comments were raised regarding: capacity of the current pumping system to remove interior drainage from the protected area after implementation of the recommended improvements to the existing levees; and constructability of some subsurface cutoff walls on the Armourdale unit when given the constraints of nearby utilities, existing overhead bridges, and potential HTRW concerns. Provision of additional detailed materials in the report and appendices resolved these issues with no impact on basic project formulation. No concerns remain. All ATR comments have been answered, back checked, and closed.

A separate ATR Certification Report has been prepared by the ATR Team Lead and reviewed and approved by the Flood Risk Management Planning Center of Expertise (FRM-PCX)

6.3.3 Independent External Peer Review

An Independent External Peer Review (IEPR) panel was established and managed by Battelle Institute under contract to the Corps' Flood Risk Management Planning Center of Expertise (FRM-PCX). Draft engineering and plan formulation documentation were reviewed by the panel prior to the Alternative Formulation Briefing (AFB). A second phase of review was conducted on the DFR. Seven comments were received from the panel on the DFR and were subsequently responded to by the Kansas City District and edits to this FFR made accordingly. All IEPR panel comments were successfully resolved. The IEPR panel produced a separate final report detailing the process, comments, and issue resolutions.

7 System Implementation Risk and Management

The separate Phase 1 and Phase 2 recommendations, as detailed and presented in the Interim Feasibility Report and this Final Feasibility Report, are complementary efforts that together address the existing Kansas Citys Flood Risk Management Project as a whole. The separate study efforts have maintained a consistent approach to improving performance and reliability within the system. The Interim Report recommendations have previously been authorized and are currently being implemented. It is important to recognize the overarching systems approach to metropolitan flood risk management within this Final Feasibility Report by providing an update on the current status of Phase 1 implementation and addressing any risks or management concerns that may arise from the integration of Phase 2 implementation into the current effort.

7.1 Implementation Status of Phase 1 Recommendations

Design phase investigations in several of the Phase 1 projects provided new information that served to refine the original Interim Feasibility Report assumptions and recommendations. This led to necessary changes in the technical scopes to implement the projects and achieve the intended purpose and performance improvement. The original and revised scope, and the current status, of each of the Phase 1 projects, is shown in Table 7-1.

Table 7-1: Status of Phase 1 Project Recommendations

Project	Original Recommendation	Current Recommendation	Status
North Kansas City Unit - Harlem Site	Buried collector system for underseepage control	Underseepage relief wells with temporary pumping capability	Construction complete
North Kansas City Unit – National Starch Site	Twenty underseepage relief wells and new pump station	Seven relief wells connecting to existing pump station	Construction complete
Fairfax-Jersey Creek Unit – BPU Floodwall	Structural wall reinforcement	Reduced structural wall reinforcement with underseepage relief wells and pump station modification	Wall modification contract awarded. Relief well and pump station contract in design phase.
Fairfax-Jersey Creek Unit – Jersey Creek Sheetpile Wall	Sheetpile wall replacement	Sheetpile wall stabilization and partial municipal wharf removal.	Design phase underway
East Bottoms Unit	Seventeen underseepage relief wells and collector piping	No change	Design phase underway
Argentine Unit	Full unit raise	No change	Design phase pending

Cost impacts of the changes in project implementation were offset by value engineering analysis where possible. Throughout the process of incorporating new information and updated methods, careful consideration has been given to maintaining the original purpose and intent of the Phase 1 recommendations, and preserving the desired economic and engineering outputs. The following discussions provide additional details of each project.

North Kansas City Unit - Harlem. Additional soil investigations and analysis during the design phase determined that the buried collector system was not a viable alternative for the Harlem reach. To provide the same level of intended reliability improvement, twenty-four pressure relief wells were required. To prevent the well discharge from impacting local homes and businesses, collection piping was installed with the wells. The well flows are directed to two manhole structures from which the flows can be pumped over the levee when necessary using portable pumps. The construction of this system was completed in 2012.

North Kansas City – National Starch. Additional soil investigations and analysis during the design phase determined that fewer relief wells were required than originally recommended. The lower flows resulting from reducing the well system are within the available capacity of an existing pump station in the area, eliminating the need for a new pump station. The construction of this system was completed in 2013.

Fairfax-Jersey Creek – BPU Floodwall. Subsurface investigations and analysis during the design phase determined that the existing foundation piles were of a different material than previously thought, and, in general, the condition of the existing foundation was better than assumed. A revised review of the existing conditions found that in addition to structural modifications, underseepage control is required to ensure the stability of the existing wall foundation. Adding underseepage control to the project scope reduces, but does not eliminate, the need for structural wall modification. To capture and handle flows from the relief wells an existing pump station within the BPU facility requires modification. Construction of the structural modifications was initiated in 2013. Initiation of the relief well and pump station construction is planned for 2014.

Fairfax-Jersey Creek – Sheetpile Wall. Design analysis determined that the original sheetpile wall replacement recommendation did not have the robustness required to ensure slope stability and reliability in the event of failure of the municipal wharf structure located within the project area. Revision of the design and construction methods accordingly resulted in significant project cost increases. Input from the City of Kansas City, KS, the owner of the wharf structure, revised the previous feasibility assumption regarding the need to maintain future operability of the wharf. Both of these factors led to a Value Engineering study that identified a revised project alternative to stabilize the sheetpile wall in place, in lieu of full replacement. This resulted in a cost savings versus the replacement alternative and provides the same desired degree of slope stability. Construction of these improvements is scheduled to begin in 2014.

East Bottoms Unit. Design efforts are underway for the recommended relief well alternative. At present, the recommended alternative has not changed. Construction initiation is planned for 2014.

Argentine Unit. Design efforts have not yet been initiated.

Efforts have been made to minimize unnecessary cost increases wherever possible, and will continue as design and construction efforts proceed; however, the changes in technical scopes summarized in the previous section have resulted in changes to the estimated project costs. Table 7-2 below the project costs as presented in the Interim Feasibility Report and the updated project cost estimates incorporating the current design information and construction status. In addition to technical changes, these estimates also reflect the inflation that has accrued between Fiscal Years 2006 and 2014.

Table 7-2: Updated Phase 1 Cost Estimates

Project	Interim Report *Oct 2005*	Current Cost Estimate *Oct 2013*
Design Deficiency		
North Kansas City	$ 8,200	$ 4030
FF-JC BPU	$ 7,900	$ 8014
Sub-Total	*$ 16,100*	*$ 12,044*
Reconstruction		
FF-JC Sheetpile	$ 8,800	$ 9,471
East Bottoms	$ 1,700	$ 4,492
Argentine	$ 52,900	$ 74,393
Sub-Total	*$ 63,400*	*$ 88,356*
Total Phase 1	$ 79,500	$ 100,401

Economic benefits within the Phase 1 levee units have been updated and compared to the cost estimates from Table 7-2. Table 7-3 presents the updated economic analysis for the Phase 1 recommendations.

Table 7-3: Updated Phase 1 Economic Analysis

Interim Feasibility Report Recommendations	First Cost	Annual Costs	Annual Benefits	Benefit/ Cost Ratio	Net Benefits
New Work/Reconstruction	$ 88,356	$ 4,487	$ 33,278	7.4	$ 28,792
Design Deficiency	$ 12,044	$ 560	$ 8,175	14.6	$ 7,616
Total Phase 1	$100,401	$ 5,046	$41,454	8.2	$ 36,408

$1000's, Oct 2013 Price Level, 3.5% Interest Rate

7.2 Integration of Phase 1 and 2 Implementation

The balance of flood risk management among these three Kansas River units is an important aspect of the system approach and must be maintained throughout the implementation efforts. Only one element of the Phase 1 project authorization is affected by the authorization and implementation of Phase 2, the Argentine Unit raise on the Kansas River. All other components of the Phase 1 recommended plan comprise specific reliability improvements in the Missouri River units of the overall system.

As discussed previously in Section 4.7.3 of this report, during large flood events (approaching the 0.33% event, or 300-year flood, and above), the proposed Argentine Unit raise would begin to induce damages downstream onto the Armourdale and CID Units, if they remain in their existing condition. These induced damages would be addressed and eliminated by implementation of the Phase 2 recommended plan. Without a Phase 2 approval and authorization, the implementation of the Argentine Units raise would either be delayed or eliminated to prevent these induced damages. Conversely, if the Phase 2 recommended plan were approved, authorized, and implemented, and the Argentine Unit raise were in some way delayed, there would be a potential upstream induced damage as well.

To manage the potential for induced damages within the system during construction, it is expected that the design and construction of the recommended plans from both phases will proceed on a parallel and coordinated schedule. Potential management risks in coordinating the implementation schedules of both phases are reduced by the fact that all three units are owned and operated by a single non-Federal sponsor, the Kaw Valley Drainage District.

7.3 Implementation Schedule and Cost Risks

Based on current evaluations, each of the three Kansas River units shows unacceptable reliability for the existing condition. Modifications to improve the existing condition reliability in each unit will have no impact on the other units, and are expected to be implemented prior to levee raise measures. This aspect of the implementation schedules and the realization of these benefits in each phase are fully independent. Only when the proposed levee raises in each phase are implemented is there an increased risk of induced damages among the units.

The current implementation schedule for each of the three units is indicated graphically in Figure 4. These implementation schedules were used as the basis of the current cost estimates for each unit.

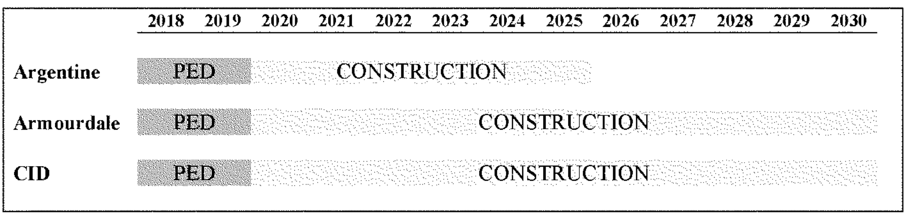


Figure 4 - Estimated Kansas River Unit Implementation Schedules

As shown, the Argentine improvements, including levee raises, are projected to be completed approximately five years before completion of the recommended Phase 2 plan. This would leave an interim period of five years in which the risk management within the system would not be balanced. If the Phase 2 implementation were delayed such that the current estimated schedule and the interim flood risk were increased, it is likely that the initiation of the Argentine levee raise would be delayed accordingly.

Changes in the schedule of levee raises in any one unit, and the effect on other units' schedules, can lead directly to cost changes due to delayed construction, increased material prices, inflation, etc. In each unit a schedule delay risk of up to ten years was identified in the cost and schedule risk analysis as very likely to occur and causing a significant impact to project cost. Such a schedule delay can occur for various reasons including availability of funds and resources, but also due to any decisions that may be made during implementation to maintain coordinated schedules between Phase 1 and 2. Any cost risk resulting from changed to either the Phase 1 or Phase 2 implementation schedule is thus captured by the individual unit estimate contingencies.

7.4 Management of Implementation Risks

Close coordination between the Non-Federal sponsor and the Kansas City District will take place throughout the implementation phase to maintain the integrated implementation schedule and identify key milestones and decision points at which schedule review and adjustments, if needed, would take place.

Based on the current schedules as presented, there is a five year period in which there is a risk of a large flood event being passed by Argentine and adversely impacting the other two units. These induced impacts occur near and above the 0.33% annual chance exceedance flood, which results in an overall 1.6% chance of occurring in the five years. While this is a relatively small risk of a large flood event, schedule changes in either phase can increase this interim risk, and may call for offsetting schedule changes in other parts of the project. Careful schedule coordination and management with the Non-Federal sponsor will ultimately determine if this type of interim risk is acceptable, and for how long.

Interim risks during construction are expected to be small and will be managed by increased risk communication and expanded emergency action planning and flood fight preparedness.

7.5 System Performance Evaluation Summary

When the study of this existing levee system began in 2000, the general thinking at the time was that all seven units in the existing system may need to be raised. Guidance resulting from the previous Reconnaissance Phase, quoted earlier in this report, directed that the study approach be based on providing a "uniform level of protection" for the system in lieu of incremental unit analyses. This was interpreted to direct the study efforts towards establishing a uniform hydraulic overtopping profile for all unit raises based upon a common design discharge for each river, and not allowing one unit to be higher or lower in height. There was also a desire by the project sponsors to have reliable 0.2%-event (500-year) protection and to address specific areas of concern observed in the 1993 flood related to underseepage, stability, or near-overtopping issues. This section provides a summary of how this system approach process was implemented throughout the formulation of recommendations, and how decisions were made throughout this study to ensure compliance with this guidance.

Missouri River Units. Initial hydraulic modeling of the existing system conditions showed that the four Missouri River units all provided acceptable overtopping margins relative to the 0.2% (500-year) event profile. Since they provided the desired level of hydraulic performance, the team and sponsor chose not to investigate raises on the Missouri River units, as and it would be illogical under the systems approach. Subsequent technical evaluations showed that even with acceptable hydraulic performance, two of these units, Fairfax-Jersey Creek and North Kansas City, had annual exceedance probabilities (AEP) less than 0.2% due to geotechnical and structural reliability deficiencies. The formulation and recommendation of alternatives for these two units focused on addressing these concerns according to applicable design criteria. The Birmingham Unit was determined fully acceptable and no further study was conducted in this unit. Statistically, the East Bottoms Unit exhibited acceptable reliability against Missouri River flooding, but underseepage issues had been observed during the 1993 flood in the Blue River tieback portion of the unit where floodwaters had peaked at 3.5 ft. below the top of levee. It was determined prudent to address a known concern that could be worse under higher loading conditions, so underseepage control alternatives were evaluated and recommended.

Kansas River Units. The initial hydraulic modeling indicated that all three Kansas River units did not pass their authorized discharge and would be overtopped by the 0.2% event, thus indicating a general need for formulation of raise alternatives in these units to improve reliability and also to be consistent with the Missouri River portion of the system. First, Argentine Unit alternatives were evaluated providing zero, three, or five feet of overtopping margin above the 0.2% event profile and addressing appurtenant geotechnical and structural modifications required at those heights. Economic benefit analysis determined that the plan based on the 0.2% event plus three feet was the NED Plan. The with-project performance of this plan was consistent with the Missouri portion of the system. Prior to publishing the Interim Feasibility Report recommending the Argentine and Missouri River unit recommendations, initial economic screening showed that plans based on the same profile also exhibited increasing net benefits for the Armourdale and CID Units over lower raise plans, and it was reasonable to expect similar performance results.

Interim and Final Reports. It was mutually agreed by HQ-USACE, Northwestern Division, the Kansas City District, and the non-Federal Sponsors to publish an Interim (or Phase 1) Report including the Missouri River unit recommendations along with the Argentine Unit on the Kansas River prior to completion of the final features and economic analyses for Armourdale and CID. Thus these two remaining units are included in this Final (or Phase 2) Report. The initial findings of Phase 1 have been subsequently confirmed by the technical and economic analyses presented in this Final Feasibility Report. The NED plan for Armourdale and CID was not determined as it would have been greater than the target established by the systems approach and the desires of the sponsors.

System Performance Conclusion. By establishing a common hydraulic profile and performance for formulation of plans in all units of the existing system, the study successfully implements the guidance directive for "uniform level of protection". As a total measure of future system performance, the Annual Exceedance Probabilities encompassing all hydraulic and engineering reliabilities are shown for each Unit in Table 7-4.

Table 7-4: System Annual Exceedance Probabilities

Unit	Existing Conditions		Future With Project	
	Median %	Expected %	Median %	Expected %
Armourdale	3.50	3.69	0.12	0.14
CID	0.33	0.47	0.12	0.19
Argentine	1.10	1.34	0.12	0.17
East Bottoms	1.40	0.19	0.10	0.10
North Kansas City	0.40	0.54	0.14	0.19
Fairfax-Jersey Creek	0.57	0.71	0.10	0.12
Birmingham	0.10	0.13	NA	NA

7.6 System Implementation Conclusion

Each phase of this feasibility study has produced recommendations for improvement within the existing system that are technically complete and effective, acceptable to the Non-Federal sponsors and the public, economically justified, and that minimize adverse impacts to the natural environmental and the existing community infrastructure. Together the Interim Feasibility Report and this Final Feasibility Report represent a comprehensive and coordinated approach to the improvement of the reliability and performance of the overall metropolitan system. While there are inherent residual risks in any flood risk management project, the specific risks unique to this project have been addressed and will be managed through the on-going partnership between the Corps of Engineers and the local project sponsors.

8 References

USACE, 2006a. Review of Completed Project, Kansas Citys Levees, Missouri and Kansas, Interim Feasibility Report, USACE Kansas City District, August 2006

USACE, 2006b. Final Environmental Impact Statement, Kansas Citys, Missouri and Kansas Flood Damage Reduction Study, Missouri and Kansas Rivers, USACE Kansas City District and U. S. Environmental Protection Agency Region VII, August 2006

Both documents above can be found at: http://www.nwk.usace.army.mil/Missions/CivilWorks/CivilWorksProgramsandProjects/KansasCitys,FloodRiskManagement.aspx

9 Recommendation

Upon considering the economic, environmental, social, and engineering aspects of making improvements to the existing Kansas Citys Project, Armourdale and Central Industrial District Units, it has been determined that a project to reduce the risk of flooding is in the public interest. Accordingly, the Corps of Engineers recommends that the Recommended Plan, as described in this report, be submitted to Congress for implementation with such modifications as the Chief of Engineers may find advisable, and in accordance with existing cost sharing and financing requirements.

The estimated implementation cost of the Final Feasibility Report Recommended Plan is $203,711,300 Federal and $109,690,700 Non-Federal for a total estimated cost of $313,402,000 at October 2013 price levels. The net benefits of the Recommended Plan are $39,966,900, indicating a very strong contribution to the nation's economic output by the project. The average annual flood risk management benefits of the Recommended Plan exceed the average annual cost by a ratio of 3.4 to 1.

All items included in the Recommended Plan are necessary to continue providing the flood risk management benefits as intended by Congress.

Federal implementation of the recommended project would be subject to the non-Federal sponsor agreeing to comply with applicable Federal laws and policies, including but not limited to:

a. Provide a minimum of 35 percent, but not to exceed 50 percent of total project costs as further specified below:

    1. Provide the required non-Federal share of design costs in accordance with the terms of a design agreement entered into prior to commencement of design work for the project;
    2. Provide, during the first year of construction, any additional funds necessary to pay the full non-Federal share of design costs;
    3. Provide, during construction, a contribution of funds equal to 5 percent of total project costs;
    4. Provide all lands, easements, and rights-of-way, including those required for relocations, the borrowing of material, and the disposal of dredged or excavated material; perform or ensure the performance of all relocations; and construct all improvements required on lands, easements, and rights-of-way to enable the disposal of dredged or excavated material all as determined by the Government to be required or to be necessary for the construction, operation, and maintenance of the project;
    5. Provide, during construction, any additional funds necessary to make its total contribution equal to at least 35 percent of total project costs;

b. Shall not use funds from other Federal programs, including any non-Federal contribution required as a matching share therefore, to meet any of the non-Federal obligations for the project unless the Federal agency providing the funds verifies in writing that such funds are authorized to be used to carry out the Project;

c. Not less than once each year, inform affected interests of the extent of protection afforded by the project;

d. Agree to participate in and comply with applicable Federal floodplain management and flood insurance programs;

e. Comply with Section 402 of the Water Resources Development Act of 1986, as amended (33 U.S.C. 701b-12), which requires a non-Federal interest to prepare a floodplain management plan within one year after the date of signing a project cooperation agreement, and to implement such plan not later than one year after completion of construction of the project;

f. Publicize floodplain information in the area concerned and provide this information to zoning and other regulatory agencies for their use in adopting regulations, or taking other actions, to prevent unwise future development and to ensure compatibility with protection levels provided by the project;

g. Prevent obstructions or encroachments on the project (including prescribing and enforcing regulations to prevent such obstructions or encroachments) such as any new developments on project lands, easements, and rights-of-way or the addition of facilities which might reduce the level of protection the project affords, hinder operation and maintenance of the project, or interfere with the project's proper function;

h. Comply with all applicable provisions of the Uniform Relocation Assistance and Real Property Acquisition Policies Act of 1970, Public Law 91-646, as amended (42 U.S.C. 4601-4655), and the Uniform Regulations contained in 49 CFR Part 24, in acquiring lands, easements, and rights-of-way required for construction, operation, and maintenance of the project, including those necessary for relocations, the borrowing of materials, or the disposal of dredged or excavated material; and inform all affected persons of applicable benefits, policies, and procedures in connection with said Act;

i. For so long as the project remains authorized, operate, maintain, repair, rehabilitate, and replace the project, or functional portions of the project, including any mitigation features, at no cost to the Federal Government, in a manner compatible with the project's authorized purposes and in accordance with applicable Federal and State laws and regulations and any specific directions prescribed by the Federal Government;

j. Give the Federal Government a right to enter, at reasonable times and in a reasonable manner, upon property that the non-Federal sponsor owns or controls for access to the project for the purpose of completing, inspecting, operating, maintaining, repairing, rehabilitating, or replacing the project;

k. Hold and save the United States free from all damages arising from the construction, operation, maintenance, repair, rehabilitation, and replacement of the project and any betterments, except for damages due to the fault or negligence of the United States or its contractors;

l. Keep and maintain books, records, documents, or other evidence pertaining to costs and expenses incurred pursuant to the project, for a minimum of 3 years after completion of the accounting for which such books, records, documents, or other evidence are required, to the extent and in such detail as will properly reflect total project costs, and in accordance with the standards for financial management systems set forth in the Uniform Administrative Requirements for Grants and Cooperative Agreements to State and Local Governments at 32 Code of Federal Regulations (CFR) Section 33.20;

m. Comply with all applicable Federal and State laws and regulations, including, but not limited to: Section 601 of the Civil Rights Act of 1964, Public Law 88-352 (42 U.S.C. 2000d) and Department of Defense Directive 5500.11 issued pursuant thereto; Army Regulation 600-7, entitled "Nondiscrimination on the Basis of Handicap in Programs and Activities Assisted or Conducted by the Department of the Army"; and all applicable Federal labor standards requirements including, but not limited to, 40 U.S.C. 3141- 3148 and 40 U.S.C. 3701 – 3708 (revising, codifying and enacting without substantial change the provisions of the Davis-Bacon Act (formerly 40 U.S.C. 276a *et seq.*), the Contract Work Hours and Safety Standards Act (formerly 40 U.S.C. 327 *et seq.*) and the Copeland Anti-Kickback Act (formerly 40 U.S.C. 276c *et seq.*);

n. Perform, or ensure performance of, any investigations for hazardous substances that are determined necessary to identify the existence and extent of any hazardous substances regulated under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), Public Law 96-510, as amended (42 U.S.C. 9601-9675), that may exist in, on, or under lands, easements, or rights-of-way that the Federal Government determines to be required for construction, operation, and maintenance of the project. However, for lands that the Federal Government determines to be subject to the navigation servitude, only the Federal Government shall perform such investigations unless the Federal Government provides the non-Federal sponsor with prior specific written direction, in which case the non-Federal sponsor shall perform such investigations in accordance with such written direction;

o. Assume, as between the Federal Government and the non-Federal sponsor, complete financial responsibility for all necessary cleanup and response costs of any hazardous substances regulated under CERCLA that are located in, on, or under lands, easements, or rights-of-way that the Federal Government determines to be required for construction, operation, and maintenance of the project;

p. Agree, as between the Federal Government and the non-Federal sponsor, that the non-Federal sponsor shall be considered the operator of the project for the purpose of CERCLA liability, and to the maximum extent practicable, operate, maintain, repair, rehabilitate, and replace the project in a manner that will not cause liability to arise under CERCLA; and

q. Comply with Section 221 of Public Law 91-611, Flood Control Act of 1970, as amended (42 U.S.C. 1962d-5b), and Section 103(j) of the Water Resources Development Act of 1986, Public Law 99-662, as amended (33 U.S.C. 2213(j)), which provides that the Secretary of the Army shall not commence the construction of any water resources project or separable element thereof, until each non-Federal interest has entered into a written agreement to furnish its required cooperation for the project or separable element.

This recommendation is contingent upon such discretionary modifications as deemed necessary by the Chief of Engineers and funding requirements satisfactory to the Administration and Congress. The recommendations contained herein reflect the information available at the time and current Departmental policies governing formulation of individual projects. They do not reflect program and budgeting priorities inherent in the formulation of a national Civil Works construction program nor the perspective of higher review levels within the Executive Branch. Consequently, the recommendation may be modified prior to implementation. However, the project partner, the States, interested Federal agencies, and other parties will be advised of any modifications and will be afforded an opportunity to comment further.

Andrew D. Sexton (date)
Colonel, Corps of Engineers
District Commander

U.S. Army Corps of Engineers, Kansas City District

Final Feasibility Report

EXHIBITS

Kansas Citys, Missouri and Kansas
Flood Risk Management Project

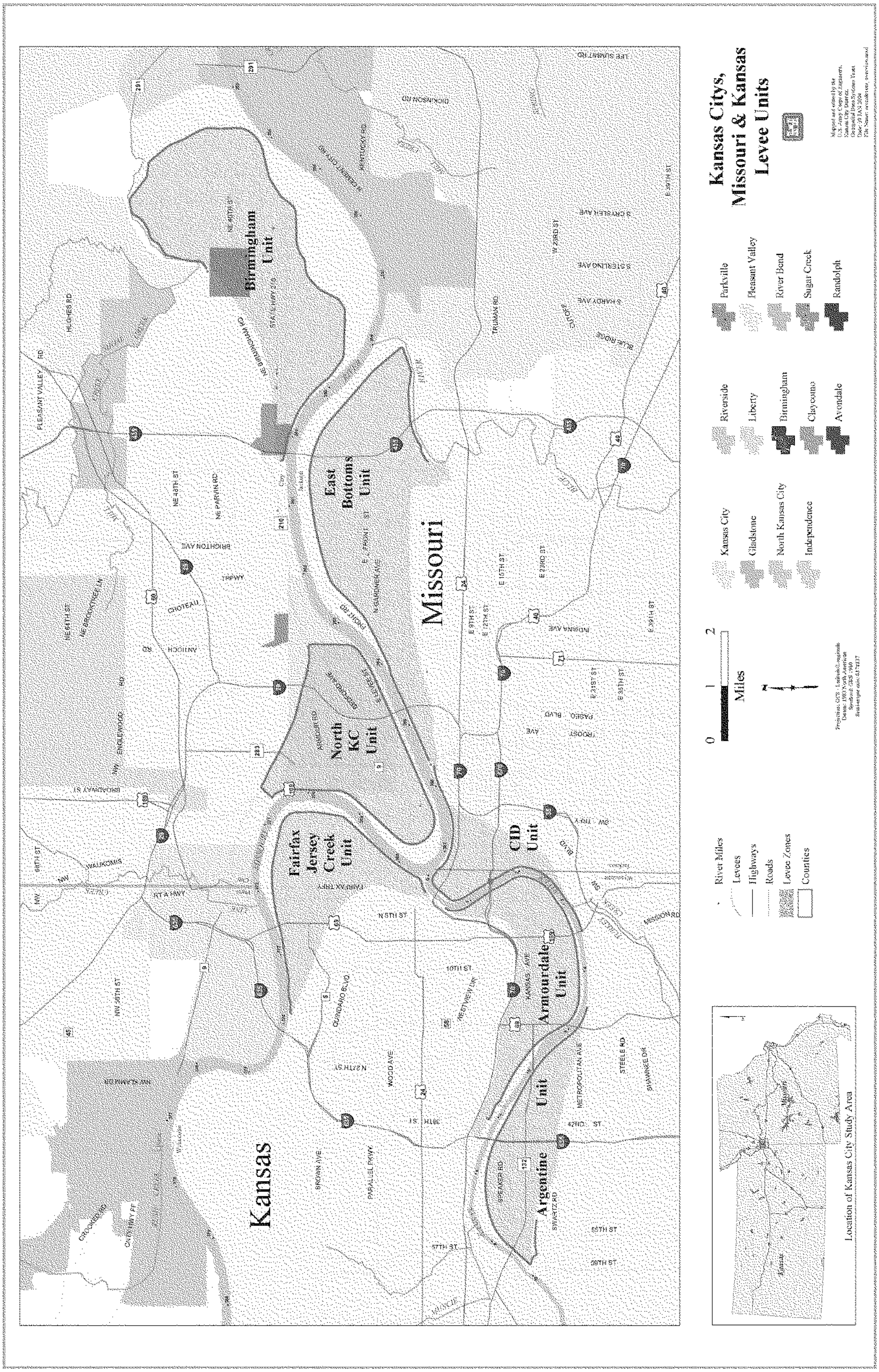

Kansas Citys, Missouri & Kansas Levee Units
EXHIBIT #1
Kansas
Missouri
Birmingham Unit
East Bottoms Unit
North KC Unit
Fairfax Jersey Creek Unit
CID Unit
Armourdale Unit
Argentine Unit
River Miles
Levees
Highways
Roads
Levee Zones
Counties
Kansas City
Gladstone
North Kansas City
Independence
Riverside
Liberty
Birmingham
Claycomo
Avondale
Parkville
Pleasant Valley
River Bend
Sugar Creek
Randolph
0
1
2
Miles
Location of Kansas City Study Area

EXHIBIT #2: Photograph of 1951 Kansas River Flood at Kansas City

- **Kansas River flood event**
- **Kansas River Basin lakes not operational**
- **All 3 Kansas River units overtopped in Kansas City**

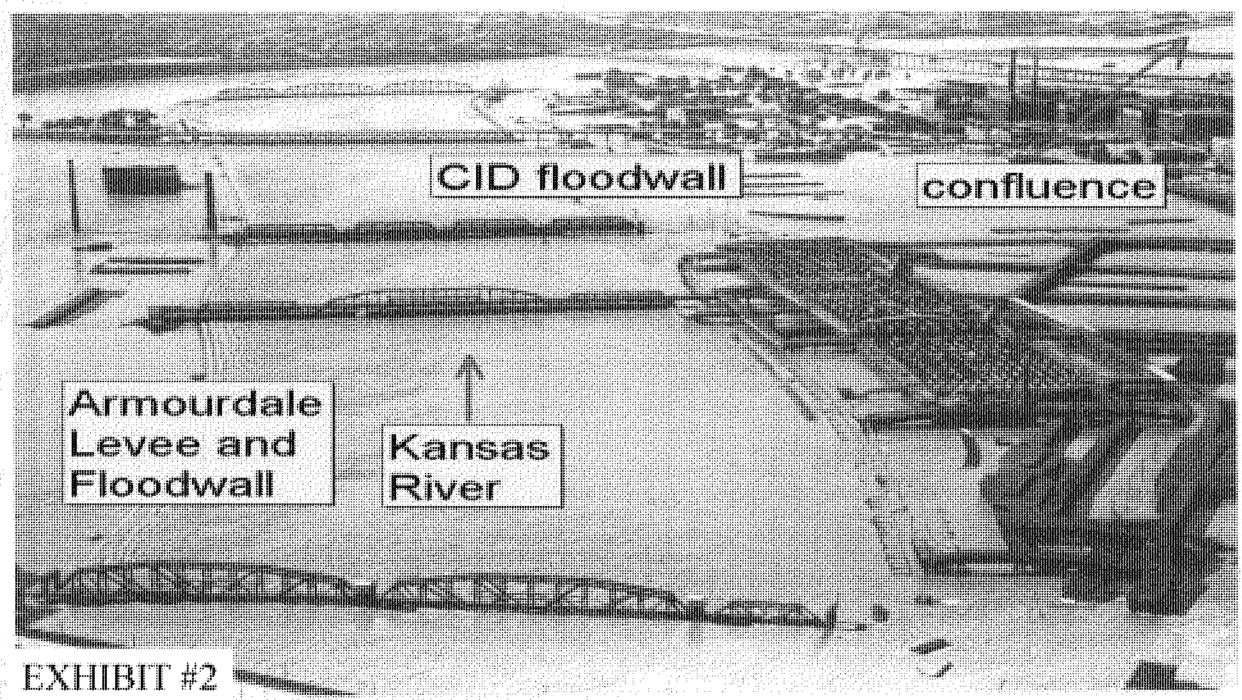
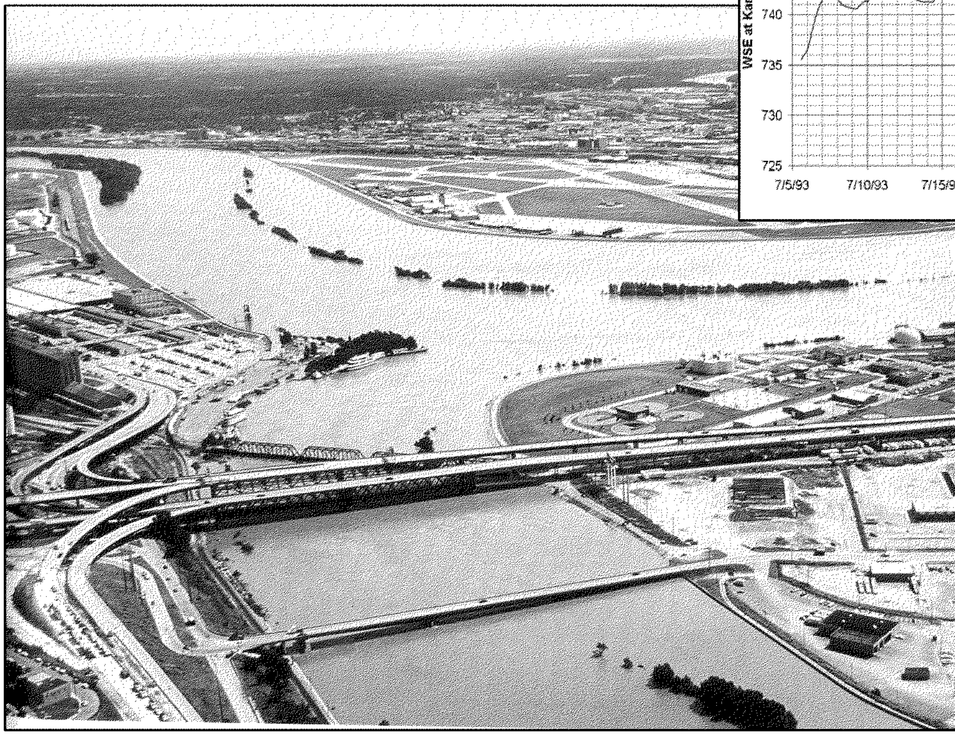
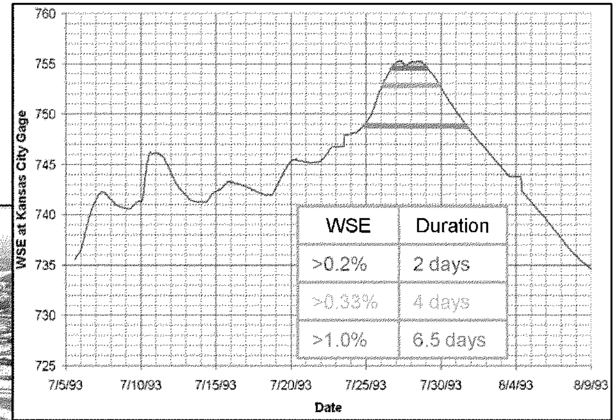


EXHIBIT #3:
1993 Flood at Kansas City

Kansas and Missouri River Confluence





Event Hydrograph
Kansas City Gage – Missouri River
WSE = Water Surface Elevation

Kansas City Levees
Armourdale Unit

Levee/Floodwall Configuration
Current and Proposed N500+3

DRAFT - WORKING COPY
Date: Jan 31, 2013

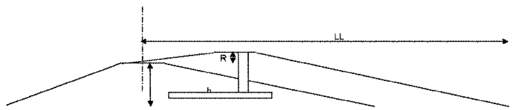
Unit Descriptions				Existing			Proposed				HTRW	Real Est. (RE)	
Map Sheet #	Station Beginning	Station Ending	Length of Unit FT	Description	Relief Well Systems, Pump Plants, and Stability Features	Primary Levee Height (h) FT	Description	Stability Control Needed	Under-seepage Control	Proposed Primary Levee Raise R=N500+3 FT	HTRW	Real Estate Notes	Notes
A1	00+05UE	3+25UE	320	Stop log Gap		13	New SLG			3.8		RR lost time issues if traffic stopped	Busy RR crossing will make replacement a challenge. Existing SLG is integral with forcemain valve.
	3+25UE	10+00UE	675	Levee		9	Riverside Levee Raise	L Berm		3.8		No RE Issues at this time	H&H input needed for this riverside Levee Raise.
	10+00UE	16+48UE	648	Levee	13+50 to 14+50: Riverside Slope Stability Berm	9.5	Modify F-AC					No RE Issues at this time. Coordination for removal of abandon RR tract required.	RR tracks are landside. Two out fall structures (Mattoon Creek is one) are riverside. Floodwall will reduce RR and outfall issues.
	16+48UE	20+08.89UE	360.89	Levee		9.5	Levee Raise			3.9		No RE Issues at this time	High Ground landside above top of existing levee.
	Station Equation: 20+08.89UE BK = 9+71.16AH												
	9+71.16	15+00	528.84	Levee		9.5	Levee Raise			3.9		No RE Issues at this time	High Ground landside above top of existing levee.
	15+00	35+00	2000	Levee		4 to 6	Levee Raise	Flattened Slope		4.1		May go beyond current right-of-way when the estimated 15 feet of permanent and 15 feet of temporary right-of-way is added. May get into abandoned rails. Looking at UP ownership.	
	35+00	42+50	750	Levee		13.5	Levee Raise	Flattened Slope w/ Berm		4.1		Will go beyond current right-of-way. Estimate up to 15 feet of permanent and 15 feet of temporary right-of-way will be needed. Will get into abandoned rails. Looking at UP ownership.	Need to confirm we can take abandoned RR - 1 set exists
A2	42+50	58+00	1550	Levee		11.5 to 13.5	T-wall on exist Levee	Flattened Slope w/ Berm		4.3	HTRW A	Potential P&G interference. RR @ toe of levee. Not in use. Need to identify the track owner. Loss of operation area and fence replacement. Fencing will have to be replaced.	1 set Rails visible but unused
	58+00	60+40	240	Levee		11.5 to 13.5	Modify F-AC				HTRW A	Area for temporary right-of-way will be limited	Floodwall to avoid Procter & Gamble
	60+40	77+80	1740	Floodwall	Osage PP Sta 76+83	11 to 15	Replace F w/ F-AC		COW starts at 62+00	4.3	HTRW A	No RE Issues at this time	Positive cut off wall to bedrock with be used for underseepage 62+00 to 82+00. Riverside earthen levee raise was considered but deemed too costly compared to floodwall. Environmental impacts are minimized with floodwall.
	77+80	81+00	320	Levee	Osage PP Sta 76+83	13	Replace L with F-AC		COW	4.6	HTRW A	Determine compensability of KAW Valley Power utilities.	Positive cut off wall to bedrock with be used for underseepage 62+00 to 82+00. KAW PP utilities will be cut off due to slurry wall. This includes all intake lines, discharge lines and associated utilities. Intake access bridge will be removed. Intake bridge is owned by KAW PP . Osage Pump Plant and Piping to remain. KAW PP Seiling weir drainage structures' and pipes exist and must be cut for installation of cutoff wall. With T-wall or levee at 77+78 would possibly interfere with operational RR tracks.
A3	81+00	96+00	1500	Levee		11 to 12	Levee Raise	Flattened Slope	COW End Sta. 82+00	4.8		No RE Issues at this time	Trash may have been dumped near area proposed for levee expansion.
	96+00	105+00	900	Levee		11	T-Wall on Levee	Flattened Slope w/ Berm		5.2		Storage Silo, Maint Bldg and Access Road possibly affected. Relocation needs to be determined. The need for estimated permanent and temporary easement will affect building, access road, and maneuver area. Land survey required to verify current easement line.	Twall to avoid KC Hardwood property. 58' to building but access to bldg is needed between levee and bldg. T-wall on bridge will jog around bridge piers. Need to route levee road to the land side to avoid 18th St. Bridge pier.
	105+00	129+00	2400	Levee		8.5 to 14	Levee Raise	Flattened Slope (w/ Berm 115 to 129)		5	HTRW B	Land survey essential to determine correct right-of-way. Both salvage yards and KC Railway will be affected by perm easement. Work area easement on KC Railyard also of concern.	
A4			12th Street Bridge Crossing				New SBG						
	129+00	157+00	2800	Levee	12th Street PP Sta 129+00; Mill St PP Sta 156+75	5.5	T-Wall on Levee	Flattened Slope		4.8	HTRW C	May go beyond current right-of-way. Estimate that will need 30 additional feet. Ditch at Station 155+00, between Trimodal and adjacent landowner to east, will need to be maintained with fill for integrity of project (this will require a permanent easement). Land survey required to verify existing easement line, encroachments and determine additional ROW required	T-wall extended upstream to reduce impacts that a levee raise would have on the 12th Street Pump Station. T-wall extended downstream to reduce impacts that a levee raise or T-wall would have on the Mill Street Pump Station. Access onto Trimodal Container Recycling has been discussed with Corps attorney and preliminary review indicates we will be able to allow contractor to drive on top of the area. This is pending a review by a Real Estate Attorney who would look into the Construction Easement issues.
A5	157+00	172+50	1550	Levee		4.5	Levee Raise	Flattened Slope	Fill ditch behind Mill St. PP	4.6	HTRW C	No RE issues at this time	
	172+50	185+00	1250	Levee		4.3	T-Wall on Levee	Flattened Slope		4.5		Appears to be within existing right-of-way except for approximately 30 feet of permanent and temporary easement requirements. Additional 30 feet will interfere with access roads, loading dock, parking, storage areas, power lines and fire hydrant.	
	185+00	197+00	1200	Levee	5th St PP Sta 185+70, W-I; Midwest Cold PP Sta 194+60	14 to 15.5	Modify F-AC		RW	4.3		No RE Issues at this time but limited RE available for work area.	Floodwall extended upstream to reduce impacts on 5th Street P.S. with a levee raise or T-wall and berm. Midwest Cold Storage building may need to be evaluated due to its old age and closeness to the levee
	197+00	206+12	912.43	Levee	W-I	10.5 to 13	Levee Raise	Flattened Slope	RW	4.3		Acq of approximately 20 ft of parking areas, storage area, and note diesel tanks at station 204+05 (this may require relocation). Land survey required to verify existing easement line, have attorney review the survey to determine any adverse possession rights entitled to landowner.	Raise Well Manholes w/ Levee Raise. Relief well collection system W-I is deep. Addition to or replacement of will be difficult.
	Station Equation 206+12.43 BK = 212+00 AH												
	212+00	227+85	1585	Levee	W-I	15	Modify F-AC		RW	4		No RE Issues at this time	Floodwall reduces impacts on adjoining businesses.

EXHIBIT #6
Page 1 of 3

Unit Descriptions				Existing			Proposed				HTRW	Real Est. (RE)	
Map Sheet #	Station Beginning	Station Ending	Length of Unit FT	Description	Relief Well Systems, Pump Plants, and Stability Features	Primary Levee Height (h) FT	Description	Stability Control Needed	Under-seepage Control	Proposed Primary Levee Raise R=N500+3 FT	HTRW	Real Estate Notes	Notes
	226+75			SLG			New SLG : KCT Bridge					RR lost time issues if traffic stopped	
A6	227+85	24000	1215	Levee	Shawnee Ave Sta 230+78; W-I	15 to 17	T-Wall on Levee	Flattened Slope w/ Berm	RW	4		Abandon RR visible. Building concerns at Sambol, SELCO and Pensky. Significant loss of parking and maneuver areas.	
	240+00	257+66.26	1766.26	Floodwall/Levee	W-I	13 to 16.5	Modify F-AC		RW	3.8		RR lost time issues if traffic stopped	CRI&P stop log gap at 249+54 has been filled in with concrete. Floodwall extended to reduce impacts on adjoining businesses having parking area and substantial loading docks. Reach includes approx. 367 lf of existing floodwall.
	Station Equation 257+66.26 BK = 257+64.97 AH												
A7	257+64.97	261+50	385.03	Floodwall	W-II	13	Modify F - New Row of Piles		Fill Slot ARW	3.5		Possible temporary lost parking due to work area easement	
	261+50	274+38	1286	Floodwall	W-II		Modify F - No New Piles		Fill Slot ARW	3.5		Possible temporary lost parking due to work area easement	
	274+36	277+21	285	SBG x 2	W-II		SLG x 2		Fill Slot ARW	3.5	HTRW D	RR lost time issues if traffic stopped	UP and MO Pac RR Bridge Closure Structure. Major coordination with RR will be needed during detailed design.
	277+21	283+50	629	Floodwall	W-II		Modify F - No New Piles		Fill Slot ARW	3			
	283+50	295+50	1200	Floodwall	W-II; KCS RR PP Sta 276+79; PBI Gordan PP Sta 286+59	12.5	Modify F - New Row of Piles		Fill Slot ARW	3	HTRW D	RR visible but not in use under bridges (slot area). Need to ID RR ownership and verify activity. Possible temporary lost of parking due to work area easement.	PBI Gordan currently expanding upstream toward the RR crossing. KVDD should watch construction activity to make sure no spoiling or disposal done in KVDD ROW. Floodwall proximity to KCS Pump Station may prove very challenging during construction.
			Cetnral Avenue Bridge Crossing				New SBG						
A8	295+50	302+58	708	Floodwall	W-III; National Beef PP Sta 295+52; Central Ave PP Sta. 299+20	17.5 to low ground	Modify F - New Row of Piles		RW	1.8		Need to ID RR ownership and verify activity. Ponding area will require a flowage easement. Two pump stations will be removed and not replaced.	A levee requires removal of pump station and well system W-III. Levee requires replacement and realignment of Cent. Ave. Off ramp. Central Ave. off ramp is in poor condition. Removal of pump stations will create a ponding area near the APAC property.
	302+58	315+00	1242	Levee		12 to 17.5	Levee Raise	Flattened Slope	RW	1.3		Temp work area may be within UP ROW	
A9	315+00	322+85.41	785.41	Levee		4 to 9	Levee Raise	Flattened Slope		1.2		Temp work area may be within UP ROW	
	Station Equation 322+85.41 BK = 39+71.83 L.E. AH												
	39+71.83LE	42+50LE	278.17	Levee		4 to 9	L	Flattened Slope		1.2		Temp work area may be within UP ROW	
	42+50LE						New SBG					RR lost time issues if traffic stopped	Since end of unit is currently high ground, the raise will end near 42+50 with a sandbag closure across the railroad.
	42+50LE	61+00LE	1850	Levee			No Raise						End authorized levee unit ~2000 feet sooner with closure structure.

Legend and Notes

**	Dimension is for entire levee footprint including stability controls on landside and riverside of levee
***	Dimension from centerline of existing levee to proposed toe includes any stability berms recommended.
L	Levee
COW	Cut off Wall for underseepage
F-AC	Floodwall with auger cast piles
ROW	Right of Way
SLG	Stop Log Gap (not all are shown)
SBG	Sand Bag Gap (not all are shown)
RW	Relief Well
W-I	Relief Well System I (Station 190+75 to 246+35) - Pumped by Shawnee Ave Pump Plant
W-II	Relief Well System II (Station 268+11 or 268+59 to 282+29) - Pumped by KCS Railroad Pump Plant
W-III	Relief Well System III (Station 296+23 to 302+40) - Pumped by Central Ave. Pump Plant
Osage	Osage Pump Station Area
RW	Relief Well
ARW	Abandon Relief Well
R	Height of Levee Raise
h	Existing Levee Height
LL	Landside Levee Width (measured from centerline of existing levee or floodwall)
HTRW A	Station 45+00 to 75+00 - Proctor and Gamble has evidence of VOCs in GW 11 & 12. Relief wells in this area may impact the contamination in the groundwater. Sta 45 to 60 - Proctor & Gamble: HTRW testing is recommended if plans call for the disturbance of soil in the fire training area.
HTRW B	Sta. 110 to 130: HTRW Testing recommended during design if plans require disturbing soil within the auto salvage yards.
HTRW C	Sta. 135 to 157: Superfund landfill and cover system adjacent to the property line. Excavation or relief wells in this area will impact the Superfund site which is not desired. Coordination with EPA is recommended for any work on site.
HTRW D	Sta. 277 to 295 - PBI Gordon: Pesticides have been found in this area. Additional investigation of the soil and groundwater is recommended prior to expansion of existing ROW limits or any relief well system.



h	existing levee height (ft)
R	proposed levee raise (ft)
LL	proposed dimension from centerline of existing levee to proposed toe (including any stability berms)

Levee/Floodwall Configuration
Current and Proposed N500+3

Unit Descriptions				Existing			Proposed					HTRW	Real Est. (RE)	
Map Sheet #	Station Beginning	Station Ending	Length of Unit FT	Description	Relief Well Systems, Pump Plants, and Stability Features	Primary Levee Height (h) FT	Description	Stability Control Needed	Under-seepage Control	Existing Centerline to Proposed Toe Dimension (LL), FT ***	Proposed Primary Levee Raise R=N500+3 FT	HTRW	Real Estate Notes	Notes
A1	83+01.29	85+00	198.71	Levee	Ohio Ave Pump Plant Sta 83+52	7.90	No Raise	na		NA	0.00		See Note A - Sheet M2	
	85+00	89+37.34	437.34	Levee		7.90	No Raise	na		NA	0.00		See Note A - Sheet M2	
	Station Equation STA 89+37.34 BK = STA 0+00 AH													
	0+00	5+00	500	Levee		0.00	No Raise	na		NA	0.00		See Note A - Sheet M2	
	5+00	10+00	500	Levee		0.00	No Raise	na		NA	0.00		See Note A - Sheet M2	
	10+00	15+00	500	Levee		0.00	No Raise	na		NA	0.00		See Note A - Sheet M2	
	15+00	18+15	315	Levee		0.00	No Raise	na		NA	0.00		See Note A - Sheet M2	
	18+15	19+73	158		Railroad Bridge								See Note A - Sheet M2	
	19+73	20+00	27	Levee		3.90	Levee Raise	na		25	0.20		See Note B - Sheet M2	Start of levee raise at station 19+73
	20+00	25+00	500	Levee		3.90	Levee Raise	na		25	0.70		See Note B - Sheet M2	
	25+00	25+90	90	Levee		3.90	Levee Raise	na		25	0.80		See Note B - Sheet M2	
	25+90	26+72.66	82.66		James Street Bridge Crossing					3.25	1		See Note A - Sheet M2	
	26+72.66	30+00	327.34	Floodwall	Cut off Wall at Sta 26+72.66	11.5	Modify Floodwall			3.25	1.1		See Note C - Sheet M2	
	30+00	32+50	250	Floodwall		11.5	Modify Floodwall			3.25	1.2		See Note C - Sheet M2	
	32+50	38+00	550	Floodwall	Mistletoe Pump Plant Sta 37+07	11.5	Modify Floodwall		Area Fill	3.25	1.3		See Note D - Sheet M2	Area Fill from Sta 32+50 to Sta 38+00, to a minimum elevation of 748, with storm drain modification.
A2	38+00	40+31.25	231.25	Floodwall		11.5	Modify Floodwall			3.25	1.4		See Note C - Sheet M2	
	40+31.25	40+61.66	30.41	Levee		10.10	Levee Raise	na		50	0.40	Advantage Metals Recycling HTRW Concern Sta 40+31.25 to Sta 51+00	See Note E - Sheet M2	The landside slope will be steepened to avoid HTRW area.
	Station Equation STA 40+61.66 BK = STA 40+91.60 AH													
	40+91.60	45+00	408.40	Levee		10.10	Levee Raise	na		50	0.40		See Note E - Sheet M2	
	45+00	50+00	500	Levee		10.10	Levee Raise	na		50	0.90		See Note E - Sheet M2	
	50+00	55+00	500	Levee		10.10	Levee Raise	na		60	1.30		See Note E - Sheet M2	
	55+00	57+07	207	Levee		10.10	Levee Raise	na		60	1.50		See Note E - Sheet M2	
	57+07	57+49	42		Central Avenue Double Deck Bridge		New SBG						See Note F - Sheet M2	New Sandbag Gap to be constructed. Construction to be coordinated with Armourdale Central Avenue Closure Structure so that Central Avenue only has to be closed once.
	57+49	60+00	251	Levee	Well Numbers 1, 2, 3, and 4 (All Plugged), KCK Flood Station #16 sta 58+35, Stock Yards #3 Pump Plant 74+21	12.10	Levee Raise	na		70	1.80		See Note G - Sheet M2	
	60+00	63+00	300	Levee		12.10	Levee Raise	na		70	2.10		See Note G - Sheet M2	
	63+00	65+00	200	Levee		11.10	Levee Raise	na	Area Fill	70	2.30	Concrete Rubble Mixed with Municipal Solid Waste	See Note H - Sheet M2	Area Fill from Sta 63+00 to Sta 74+75, to a minimum elevation of 751 at levee toe. Approx slope of fill from levee to ground is 1%
	65+00	70+00	500	Levee		11.20	Levee Raise	na		70	2.70		See Note H - Sheet M2	
A3	70+00	74+35.94	435.94	Levee		11.50	Levee Raise	na		70	2.80		See Note H - Sheet M2	
	74+35.94	75+25	89.06	Floodwall	Well Number 5 (Plugged)	12.8	Modify Floodwall			8	2.8		See Note I - Sheet M2	
	75+25	75+76	51		Missouri Pacific Railroad Bridge		New SBG						See Note I - Sheet M2	
				floodwall	Well Number 6 (Plugged)	12.8	Modify Floodwall			8	2.9			
				Sandbag Gap	Union Pacific Railroad Bridge		New SLG							
	75+76	77+27.75	151.75	Floodwall		12.8	Replace FW w/ New FW			30	3		See Note C - Sheet M2	
	77+27.75	80+00	272.25	Levee	Well Numbers 7 thru 17 (Number 7 is plugged), Gateway Pump Plant 80+90, South Stockyards 84+90, 88+19, and 94+32, Stockyards #1 98+05	13.50	Levee Raise	Replace landward retaining wall with berms	Area Fill	120	3.30		See Note H - Sheet M2	Area Fill from Sta 77+27 to Sta 94+50, to a minimum elevation of 749, with Gateway 2000 inlet Modification. Approx slope of Area fill from levee to ground is 1%
	80+00	85+00	500	Levee		13.50	Levee Raise		Area Fill	120	3.60		See Note H - Sheet M2	
	85+00	90+00	500	Levee		13.80	Levee Raise		Area Fill	120	3.70		See Note H - Sheet M2	
	90+00	95+00	500	Levee		14.50	Levee Raise		Area Fill	120	3.40		See Note H - Sheet M2	
	95+00	100+00	500	Levee		14.10	Levee Raise			120	3.20		See Note H - Sheet M2	
A4	100+00	102+73.39	273.39	Levee		4.55	Levee Raise			120	2.90		See Note H - Sheet M2	
	102+73.38	106+00	326.62	Floodwall	Abandoned CRI&P RR Bridge Sta 104+51.5	13.3	New Floodwall at RR Bridge			11.75	2.8		See Note C - Sheet M2	The Abandoned CRIP RR Bridge had a SLG that was filled in. Replace gap with new raised floodwall.
	106+00	110+00	400	Floodwall	Kemper Arena Pump Plant 106+49	13.3			Wells	11.75	2.9		See Note J - Sheet M2	27 new relief wells, surface discharging at total of 18 cfs.
	110+00	115+00	500	Floodwall		13.4	Modify Floodwall		Wells	11.75	3		See Note J - Sheet M2	
	115+00	116+70	170	Floodwall		13.5	Modify Floodwall		Wells	11.75	3.1		See Note J - Sheet M2	
	116+70	120+00	330	Floodwall		13.6	Modify Floodwall			11.75	3.2		See Note C - Sheet M2	
	120+00	125+00	500	Floodwall		13.6	Modify Floodwall			11.75	3.3		See Note C - Sheet M2	
	125+00	130+00	500	Floodwall		13.6	Modify Floodwall			11.75	3.6		See Note C - Sheet M2	
A5	132+20			SLG	KC Terminal Double Deck Hydraulic RR Bridge		Replace SLG						See Note K - Sheet M2	
	130+00	137+50	750	Floodwall		13.6	Modify Floodwall		Wells	11.75	3.8		See Note J - Sheet M2	Construct approximately 600 feet of new concrete floodwall tieback from existing wall at Station 137+50 southeasterly to the existing bluff. Install 30 new relief wells surface discharging a total of 45 cfs. Install two new stop log gaps at railroad crossings.
	137+50	BLUFF	600	Floodwall		13.7	New Floodwall		Wells	11.75			See Note G - Sheet M2	
	137+50	145+00	750	Floodwall		13.9	No Raise	na		NA	0.00			
	145+00	150+00	500	Floodwall		13.9	No Raise	na		NA	0.00			
	150+00	155+00	500	Floodwall		14.1	No Raise	na		NA	0.00			
	155+00	160+00	500	Floodwall		14.2	No Raise	na		NA	0.00			
	160+00	165+00	500	Floodwall		14.4	No Raise	na		NA	0.00			
	165+00	166+25	125	Floodwall		14.5	No Raise	na		NA	0.00			
	166+25	168+16	191	Stop log Gap	Railroad Corridor			na		NA	0.00			
	168+16	168+36	20	Levee		8.80	No Raise	na		NA	0.00			

Kansas Citys Levees
Central Industrial District Unit - Kansas

Levee/Floodwall Configuration
Current and Proposed N500+3

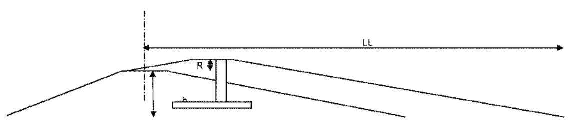
DRAFT - WORKING COPY
Date: Jan 31, 2013

LEGEND AND NOTES

Abbreviation	Description
**	Dimension is for entire levee footprint including stability controls on landside and riverside of levee
***	Dimension from centerline of existing levee to proposed toe includes any stability berms recommended.
L	Levee
FW	Floodwall
NR-FW	New Raised Floodwall
COW	Cut off Wall for underseepage
F-AC	Floodwall with auger cast piles
RAP	Railroad or Roadway above Protection
ROW	Right of Way
SLG	Stop Log Gap (not all are shown)
SBG	Sand Bag Gap (not all are shown)
RW	Relief Well
W-I	Relief Well System I (Station 190+75 to 246+35) - Pumped by Shawnee Ave Pump Plant
W-II	Relief Well System II (Station 268+11 or 268+59 to 282+29) - Pumped by KCS Railroad Pump Plant
W-III	Relief Well System III (Station 296+23 to 302+40) - Pumped by Central Ave. Pump Plant
Osage	Osage Pump Station Area
RW	Relief Well
ARW	Abandon Relief Well
R	Height of Levee Raise
h	Existing Levee Height
LL	Landside Levee Width (measured from centerline of existing levee or floodwall)
HTRW	

Station Equations
Station Equation STA 89+37.34 BK = STA 0+00 AH
Station Equation 40+61.66 BK = 40+91.60 AH

Real Estate Notes	
A	No Real Estate Concerns at this time
B	Some additional permanent ROW and 15' of temporary ROW will be required.
C	Some temporary ROW may be needed.
D	Some temporary ROW may be needed. Will need temporary construction ROW for the area fill.
E	Will need 15' additional permanent ROW and 15' temporary ROW.
F	Need to determine how much additional permanent and temporary ROW will be required.
G	Will need additional permanent ROW and 15' temporary ROW.
H	Will need additional permanent ROW and 15' temporary ROW. Will need temporary construction ROW for the area fill.
I	Need to determine if additional permanent ROW is needed and how much temporary ROW will be required.
J	Some temporary ROW may be needed. Need to determine if additional permanent ROW will be needed for the relief wells.
K	Need to determine if additional permanent ROW is required and how much temporary ROW is required.

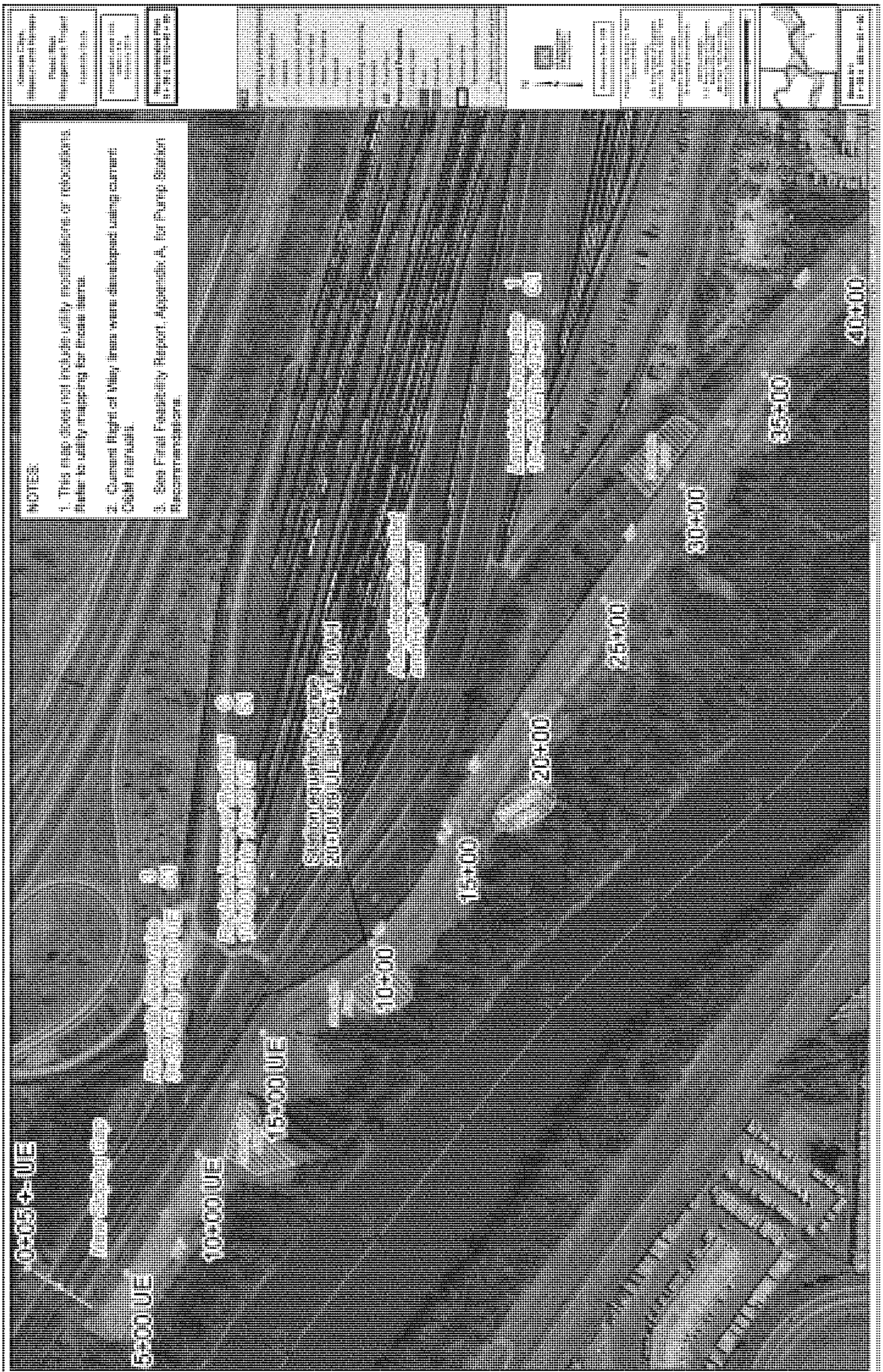


h existing levee height (ft)
R proposed levee raise (ft)
LL proposed dimension from centerline of existing levee to proposed toe (including any stability berms)

EXHIBIT #7
2 of 2

Final Feasibility Report

MAPS



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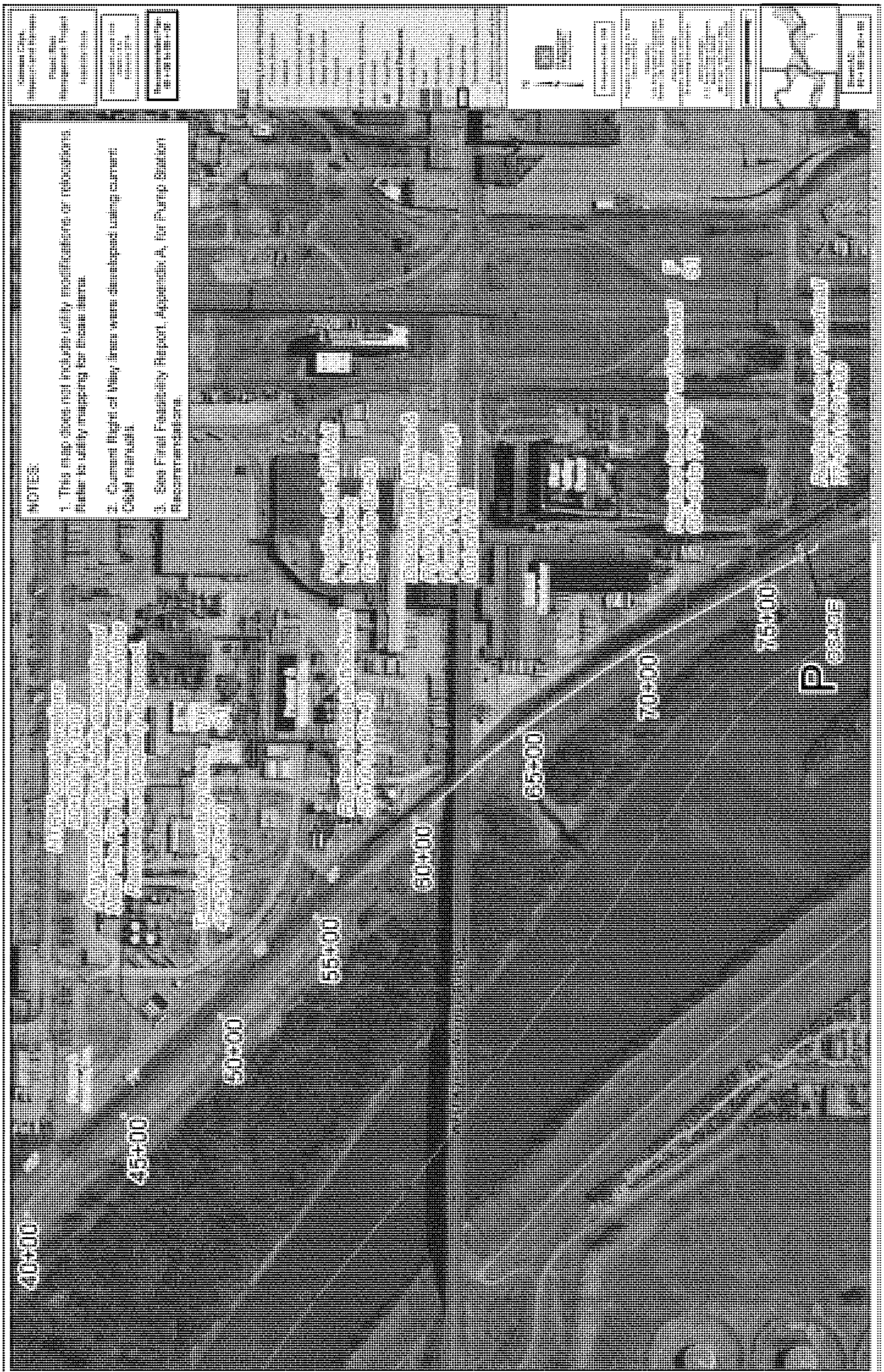
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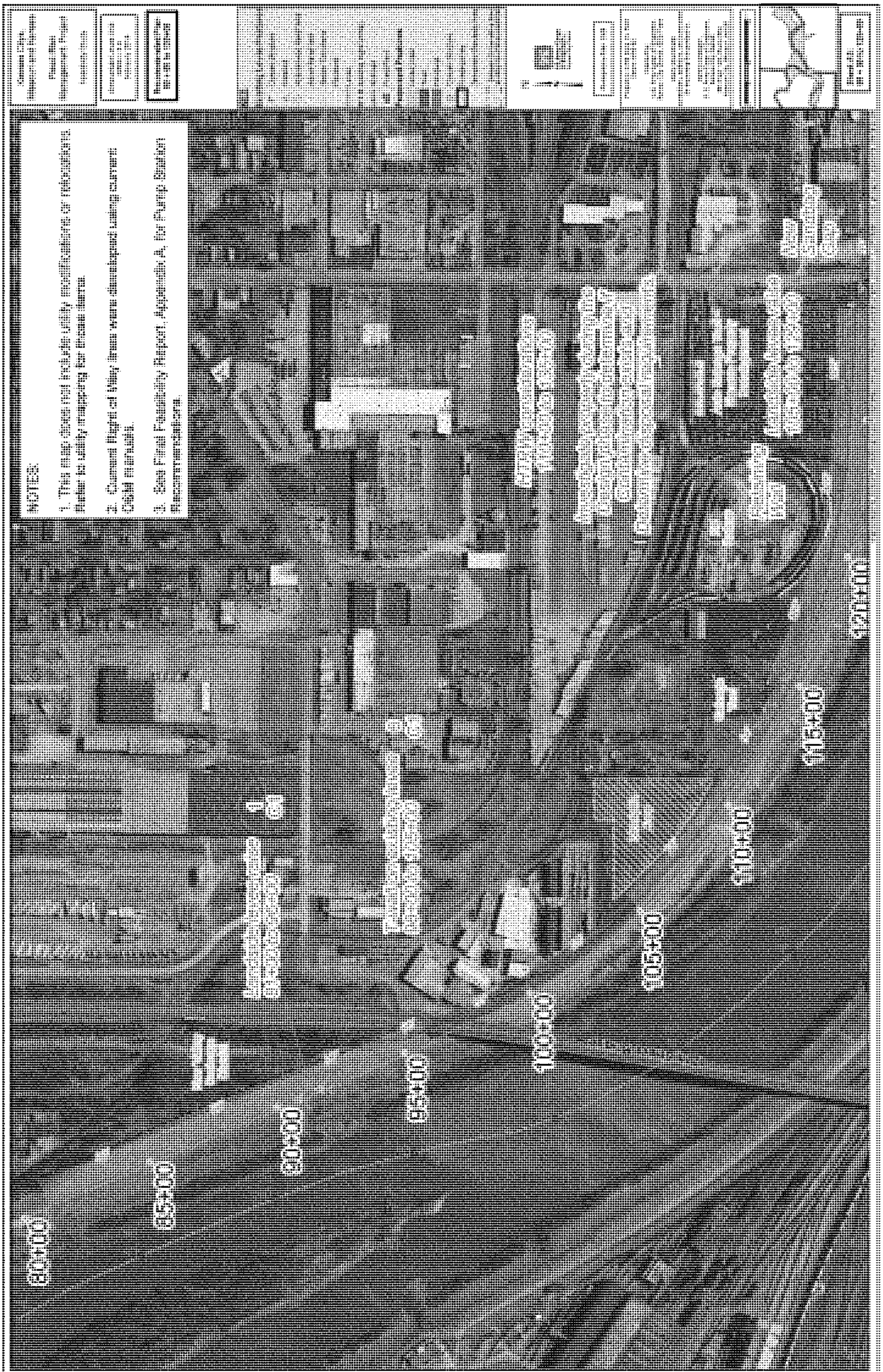

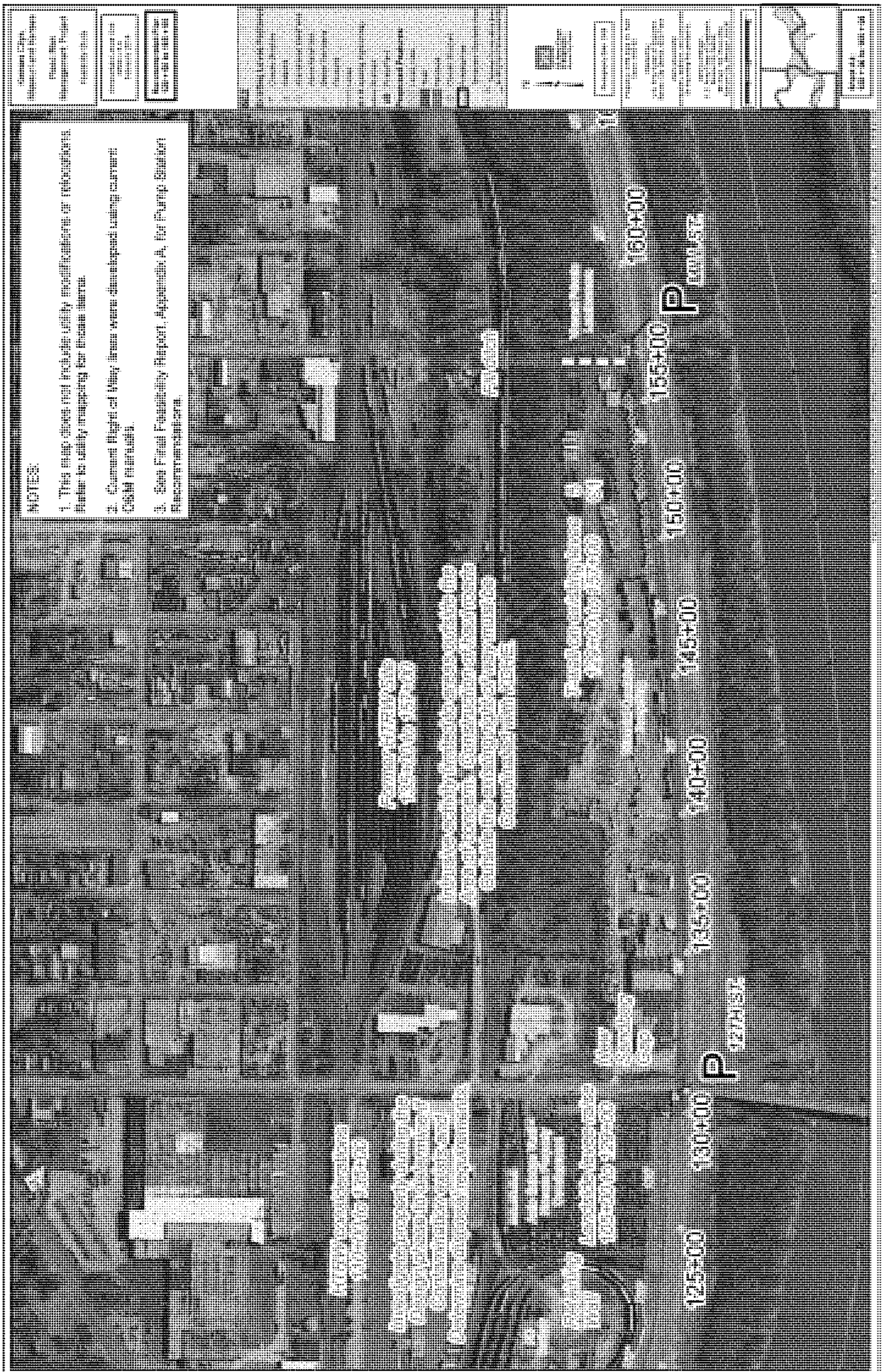
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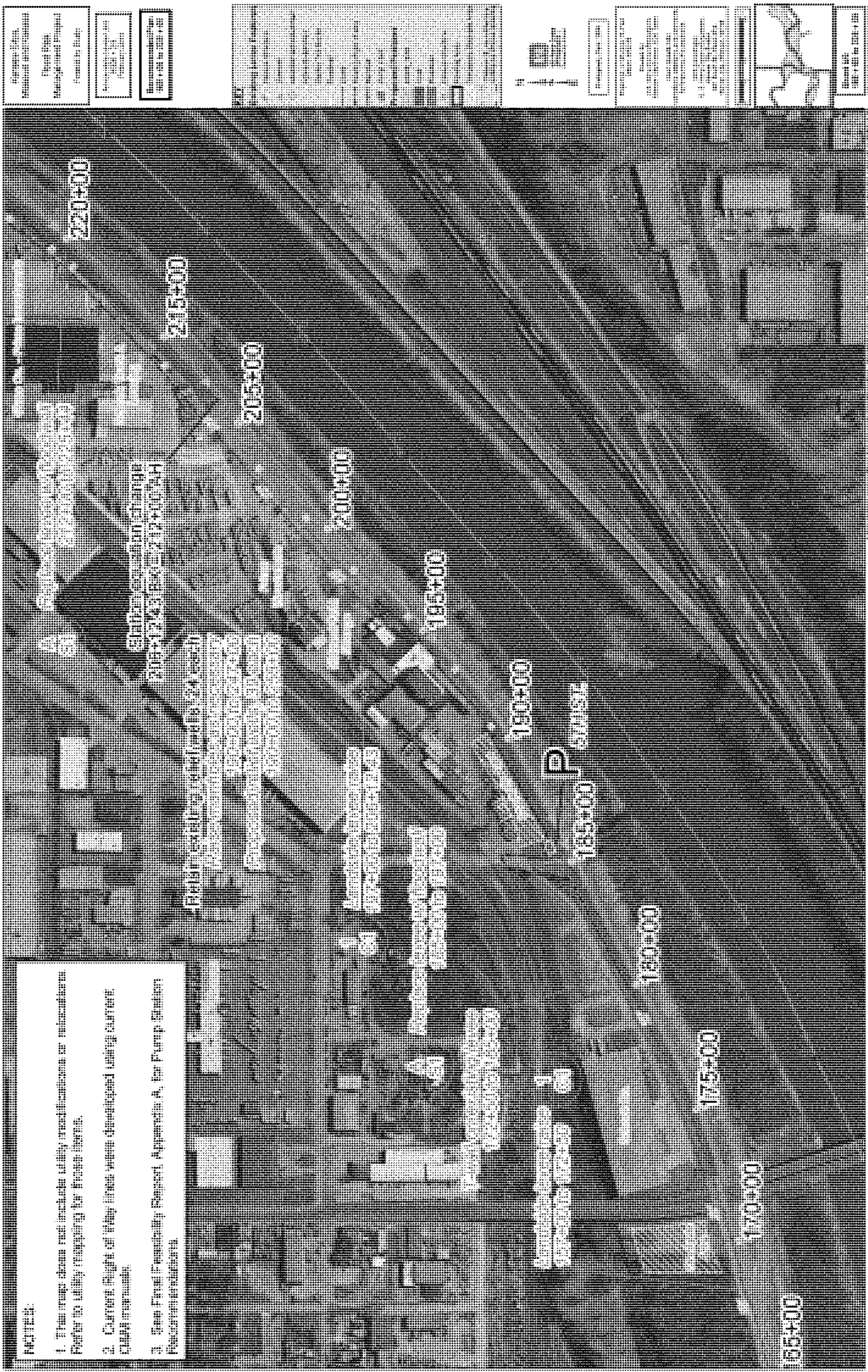
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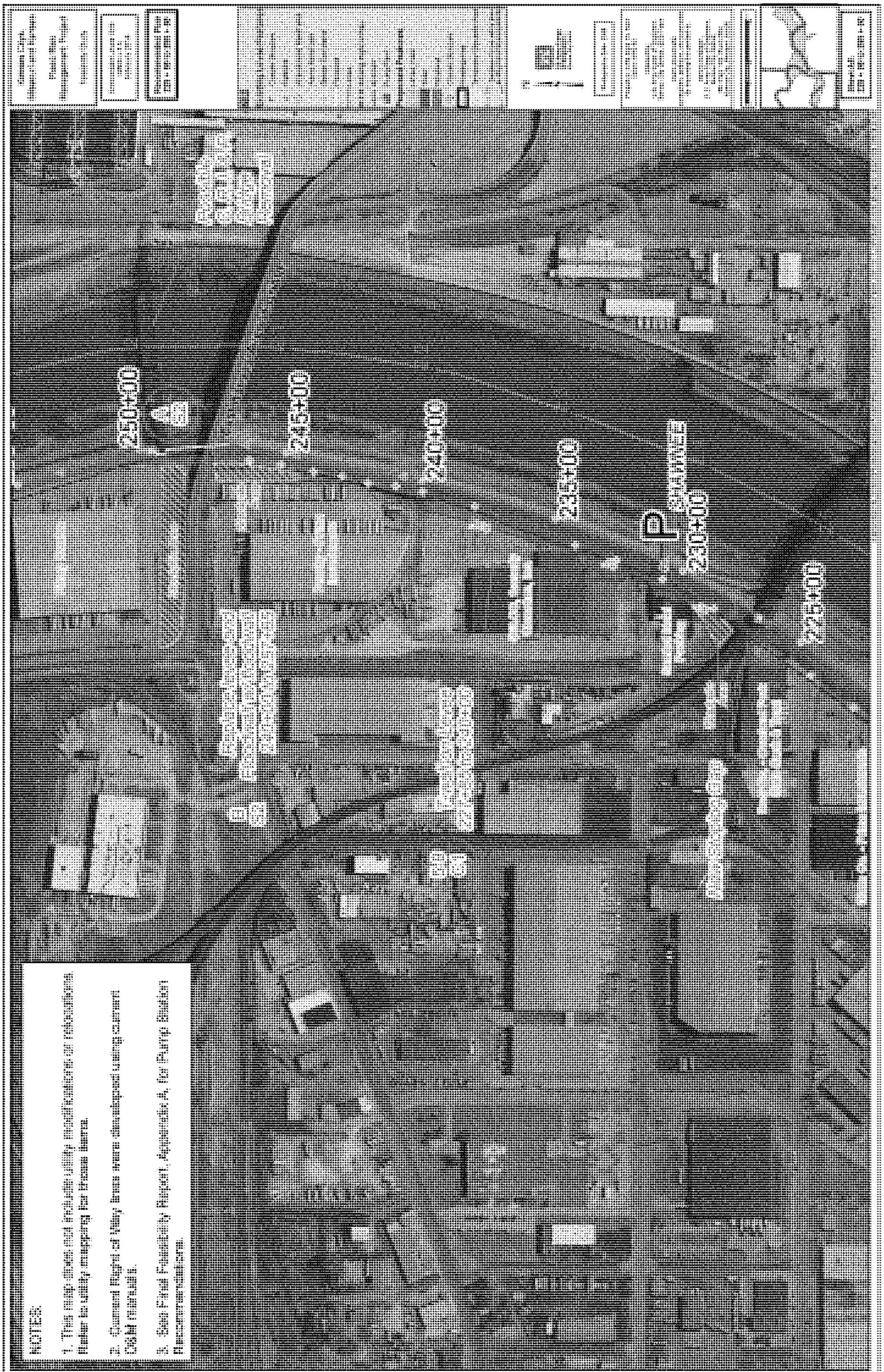
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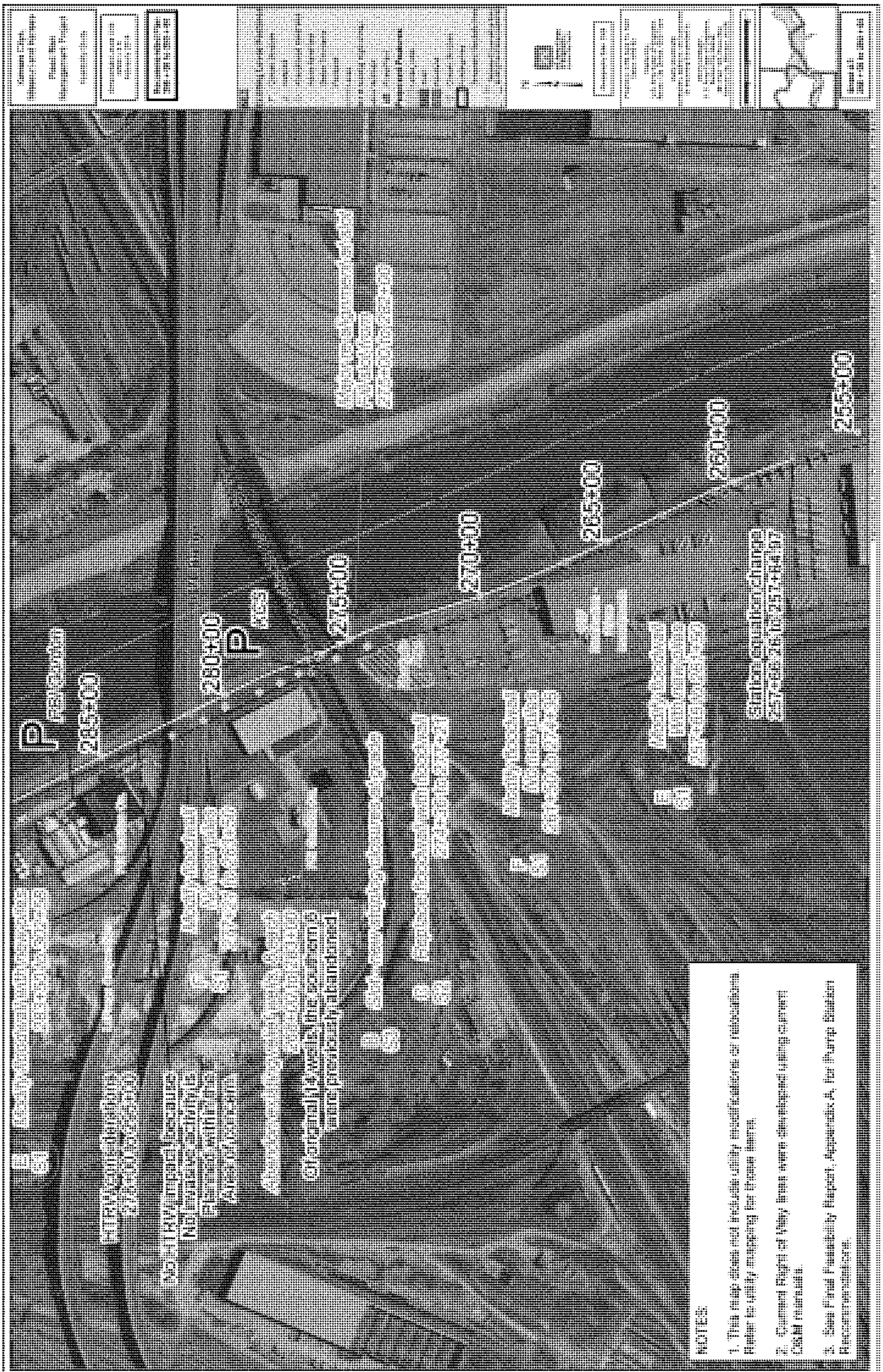
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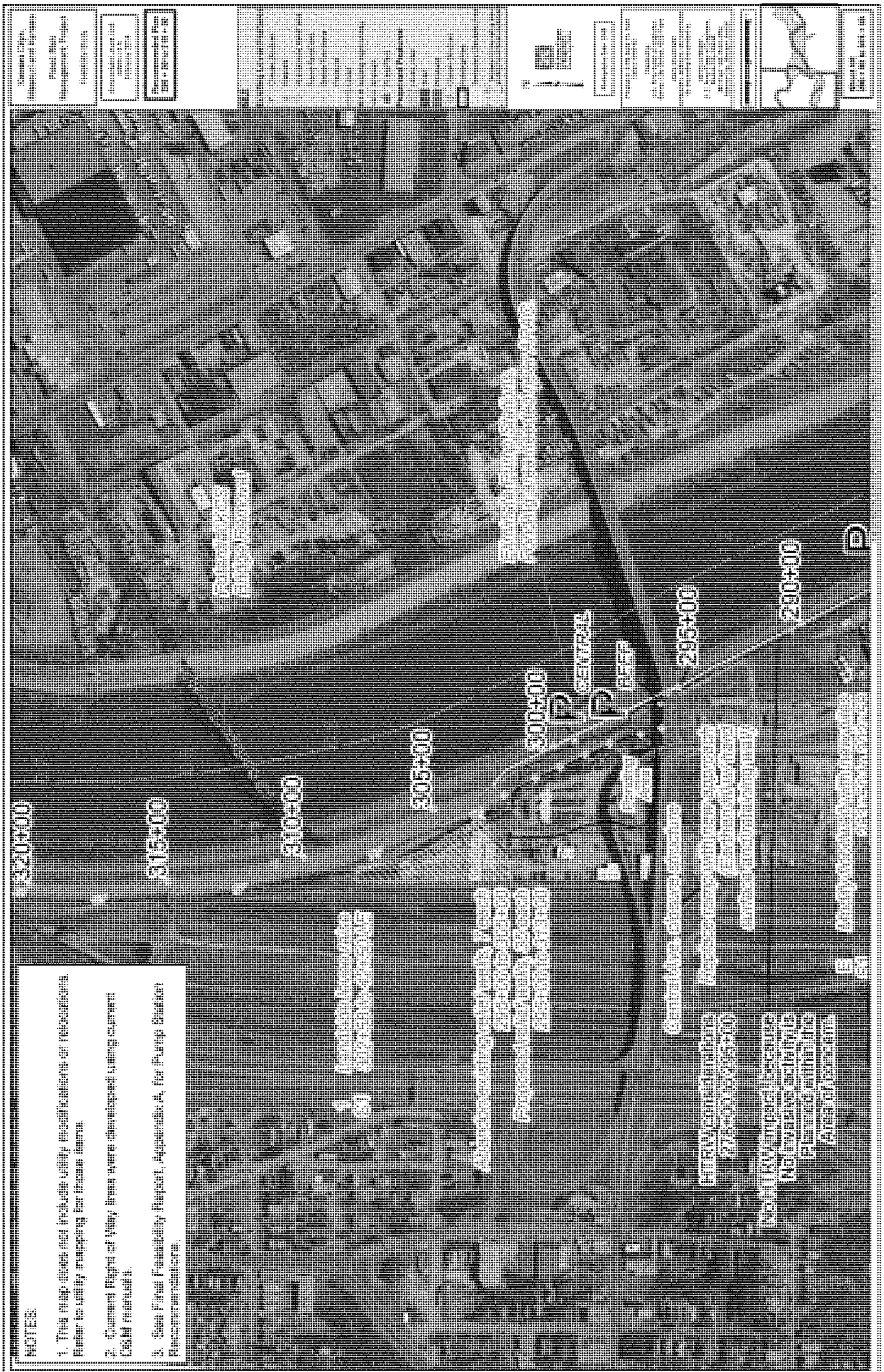
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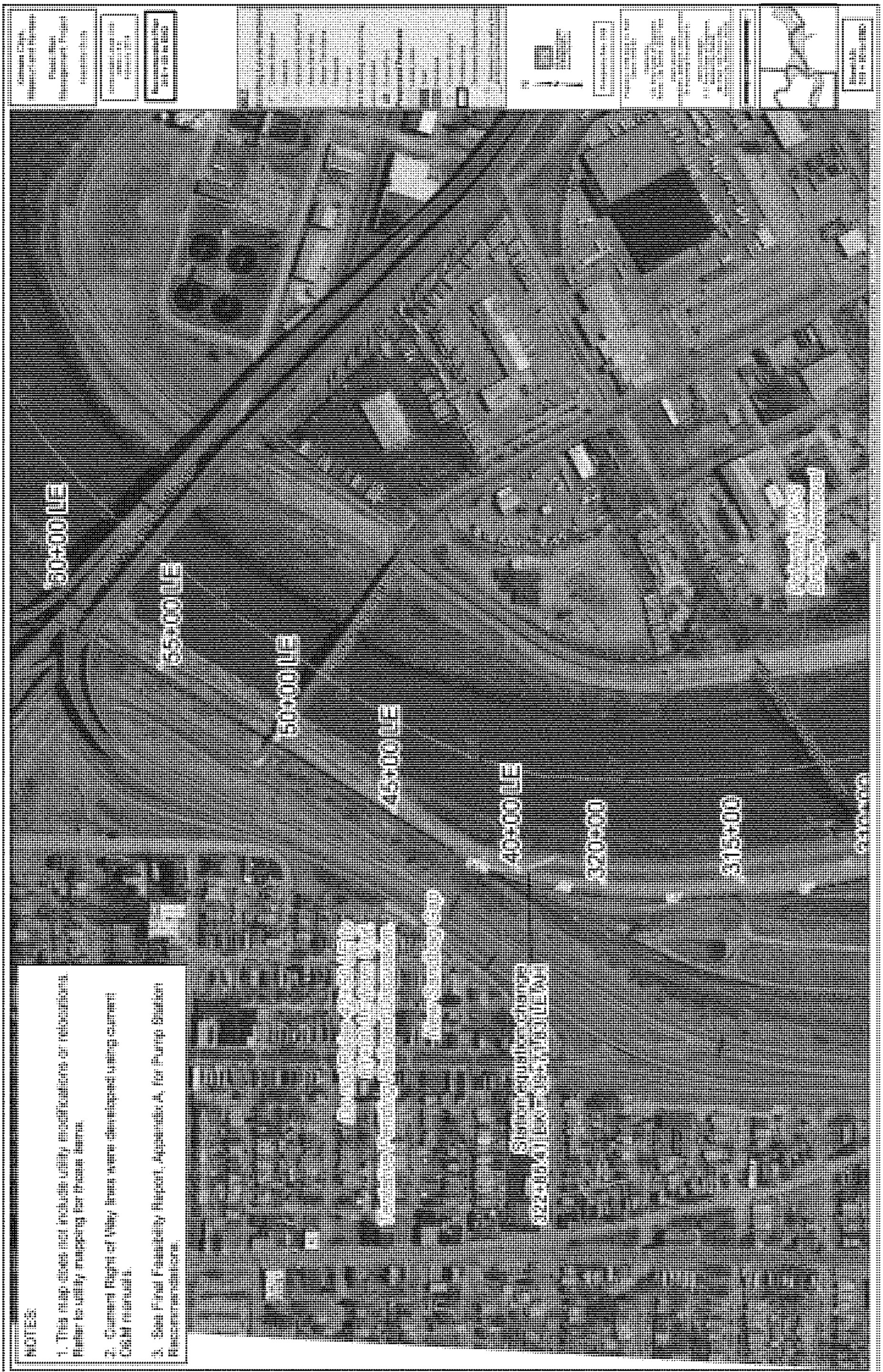
NOTES:
1. This map does not include utility modifications or relocations. Refer to utility mapping for those items.
2. Current Right of Way lines were developed using current O&M manuals.
3. See Final Feasibility Report, Appendix A, for Pump Station Recommendations.
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45+00 LE
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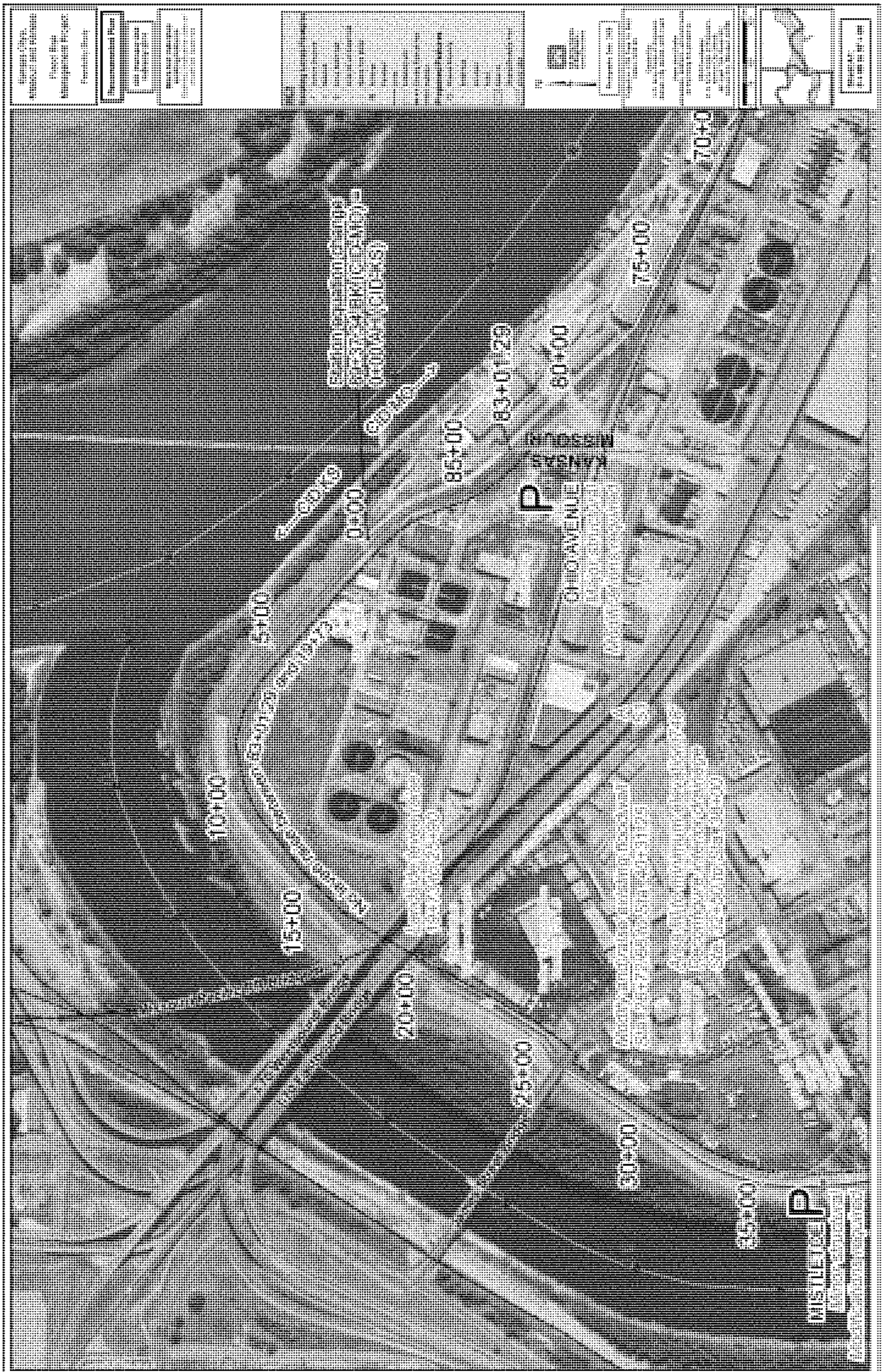
Station equation change
320+00
315+00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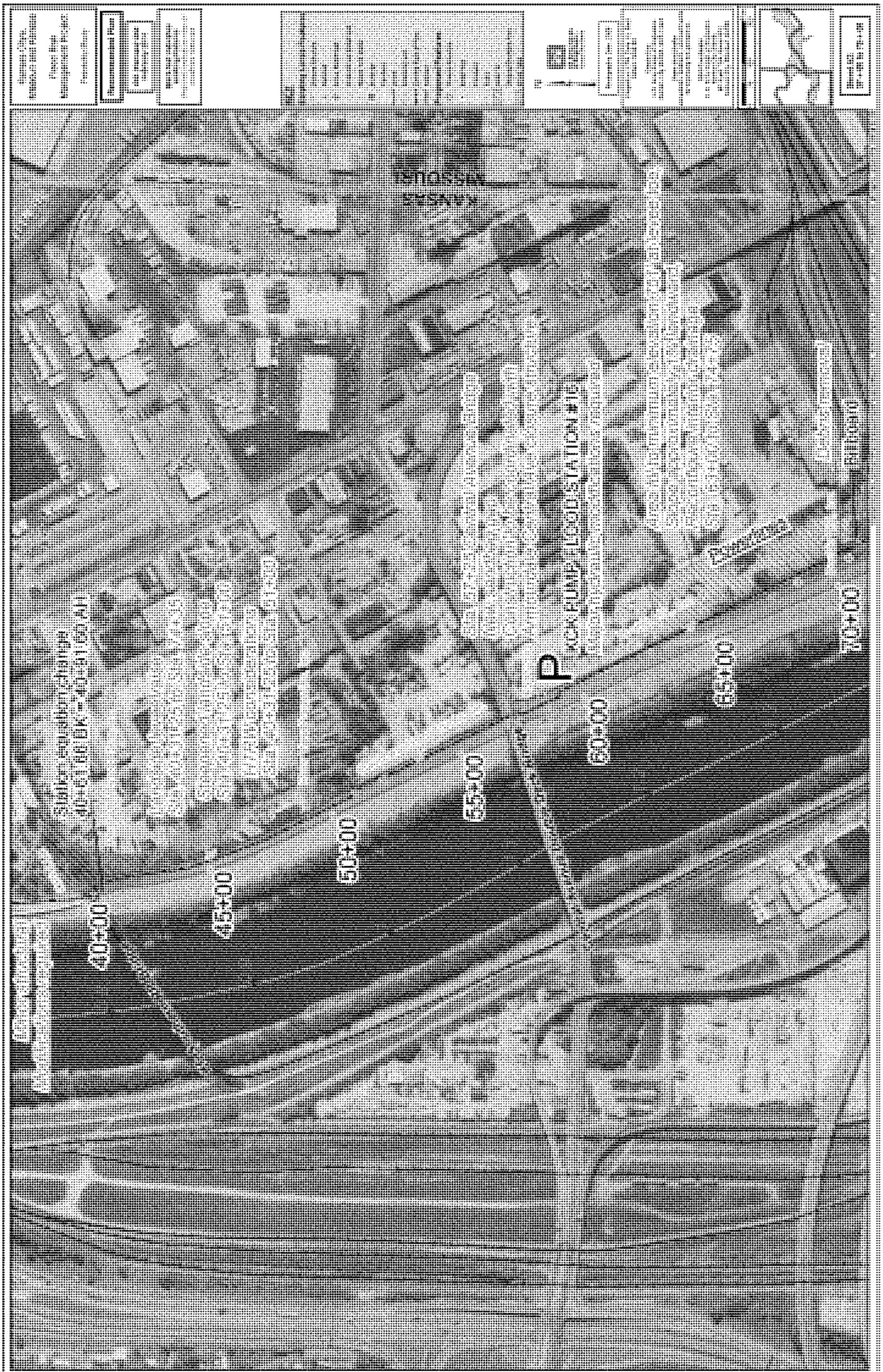
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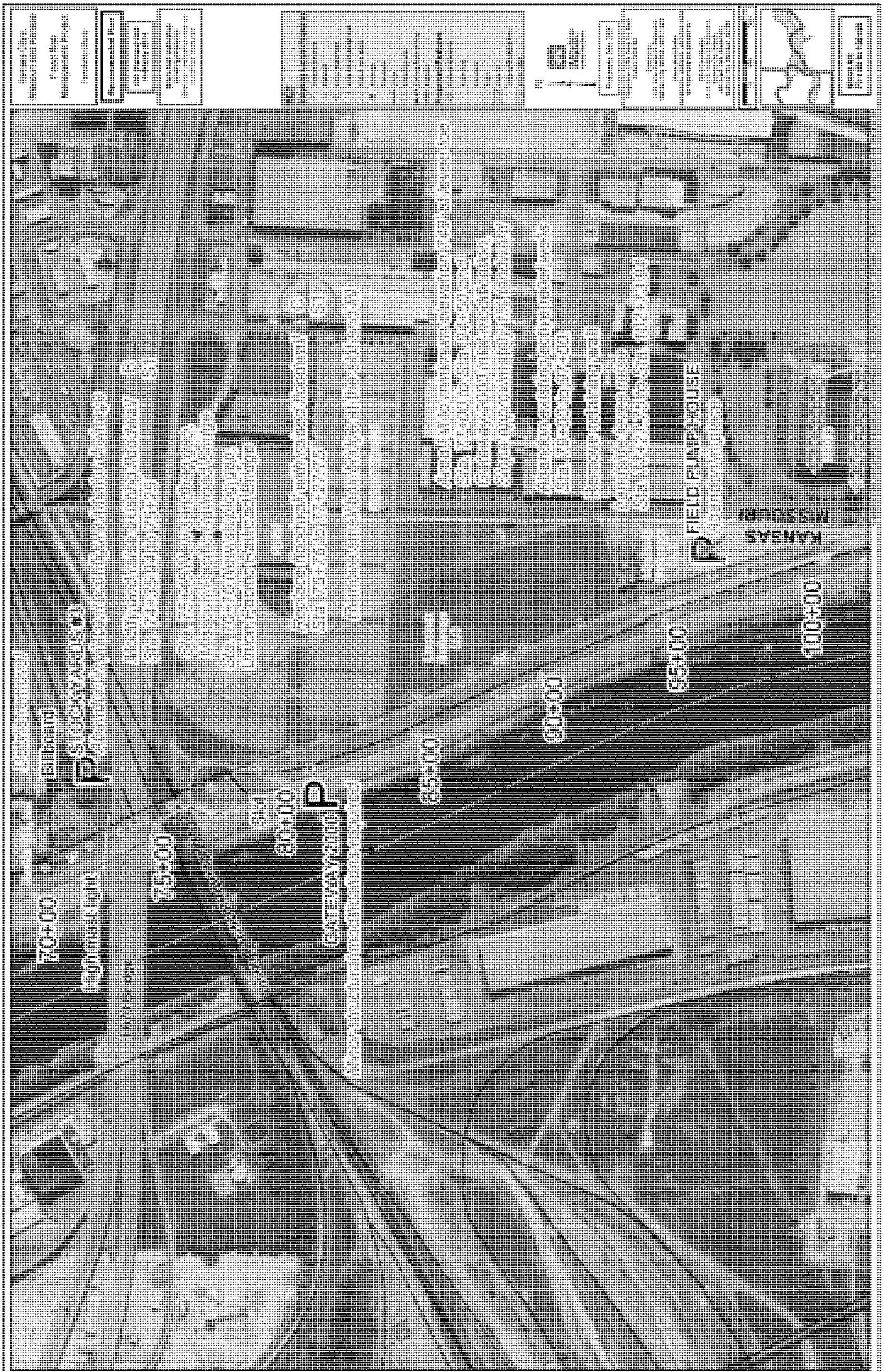
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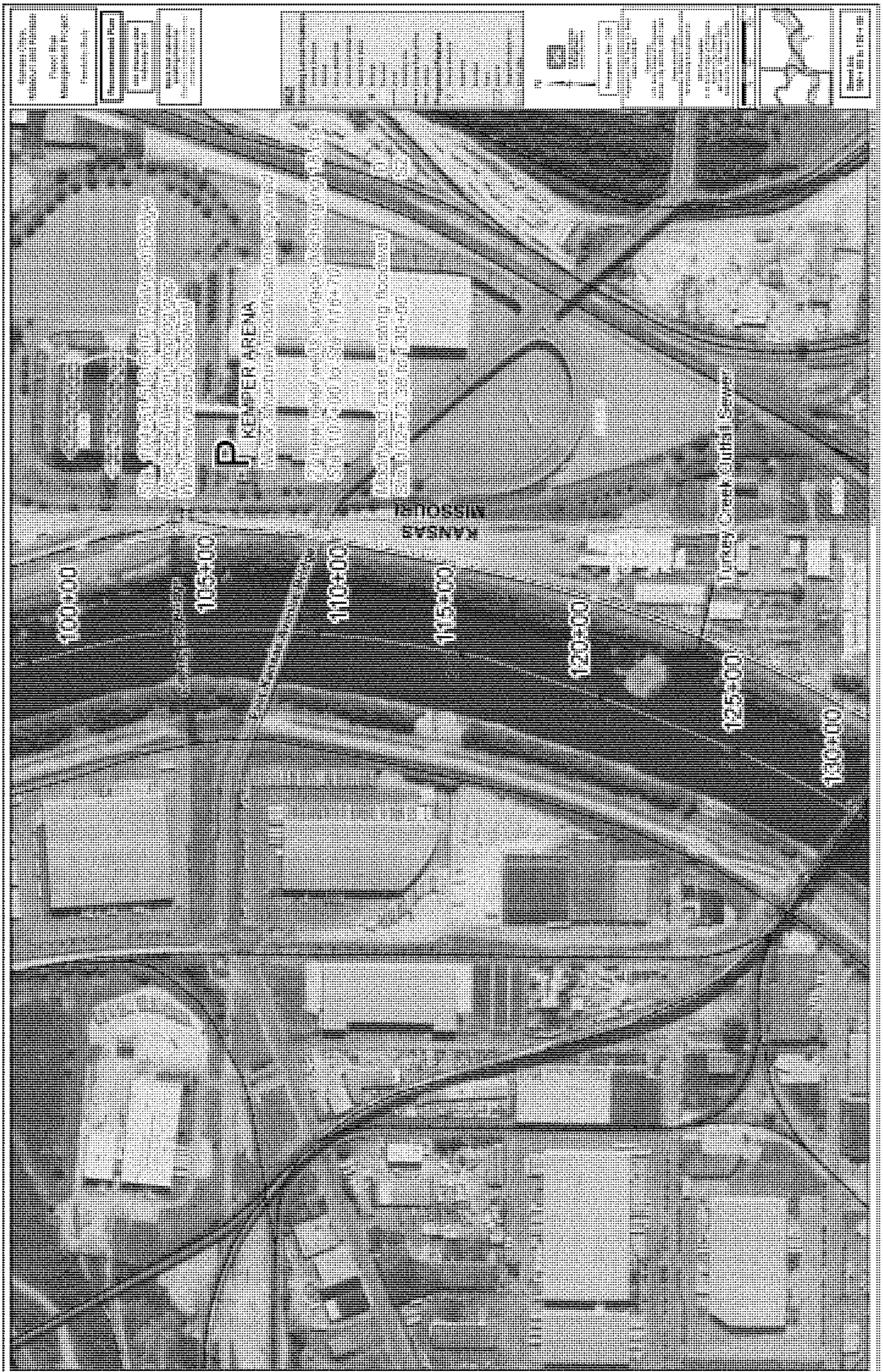
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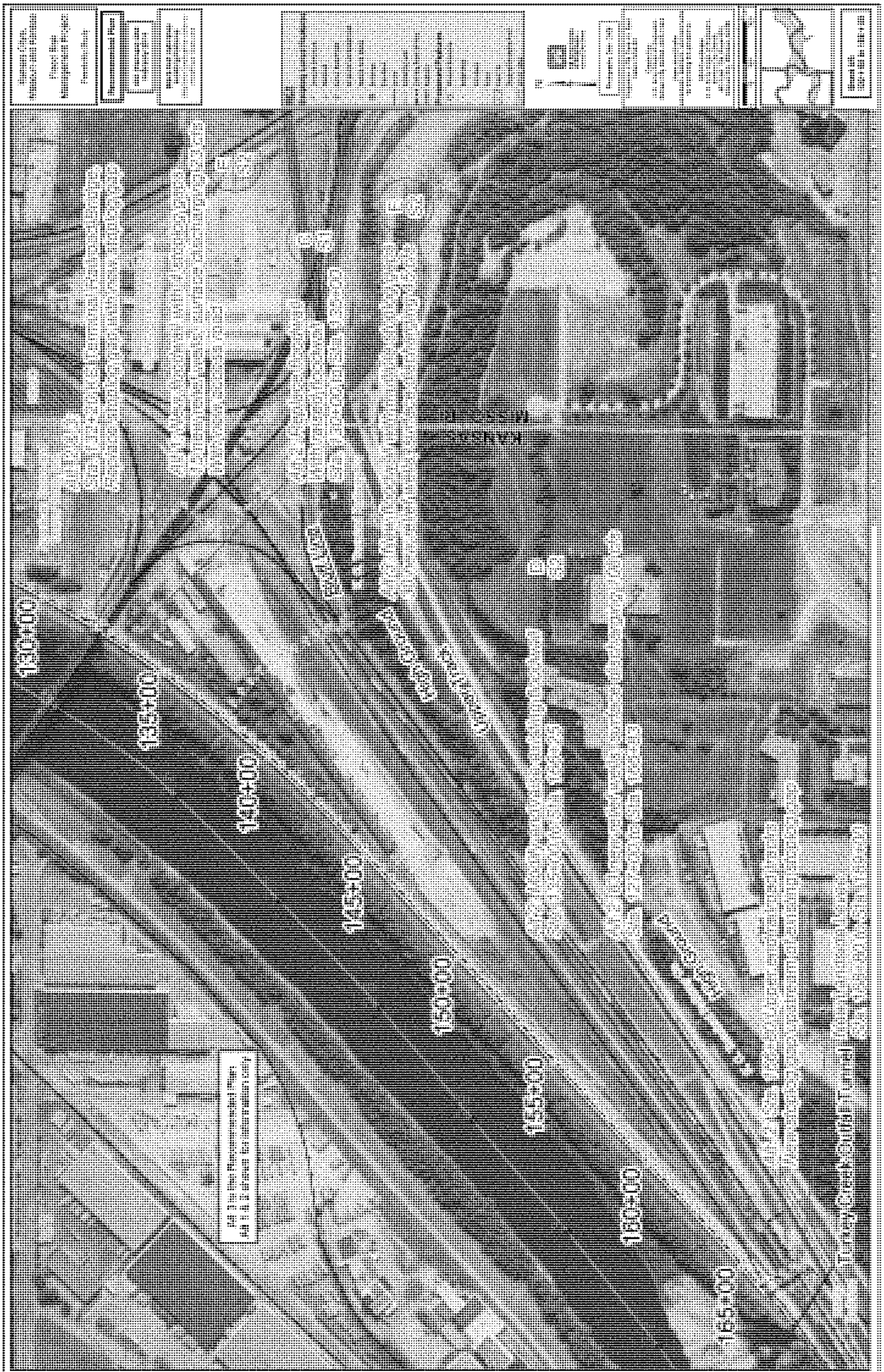
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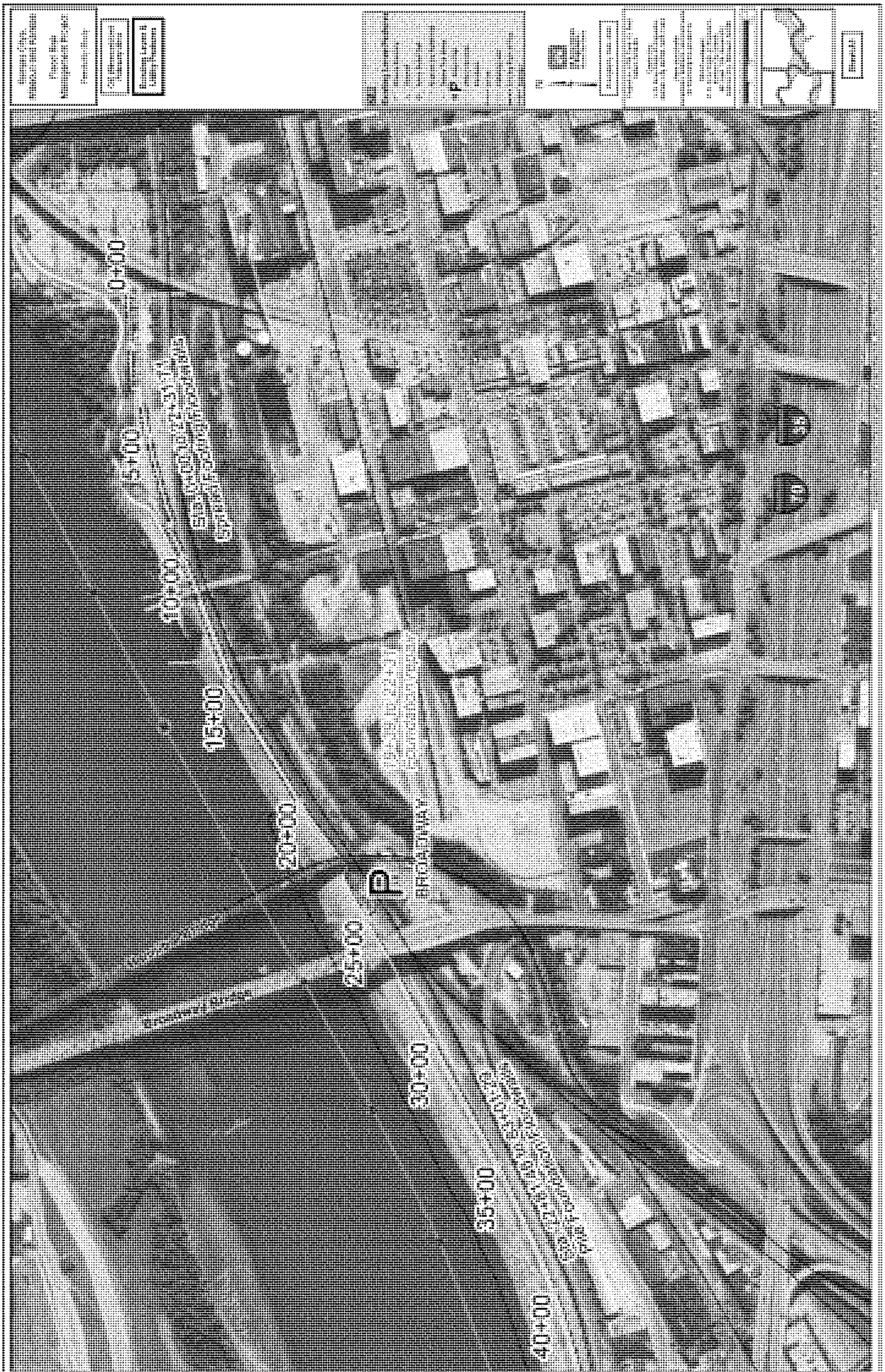
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BROADWAY

Final Feasibility Report

APPENDIX A

ENGINEERING ANALYSIS

ENGINEERING APPENDIX A
Kansas Citys, Missouri and Kansas
Flood Risk Management Project Feasibility Study

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INDEX TO ENGINEERING APPENDIX TABLES

INDEX TO ENGINEERING APPENDIX EXHIBITS

ACRONYM LIST

ACI – American Concrete Institute
AH – Ahead
APWA – American Public Works Association
A.S.B. – Armour-Swift-Burlington
BK - Back
BPU – Board of Public Utilities
CEMVS – St. Louis District
CENWK – Kansas City District
CH – Highly Plastic Clay
CID – Central Industrial District
CIP – Cast Iron Pipe
CL – Low to Moderate Plastic Clay
CMP – Corrugated Metal Pipe
COE – Corps of Engineers
COV – Coefficient of Variation
DIP – Ductile Iron Pipe
EC – Existing Conditions
EC-GD – Engineering Construction-Geotechnical Dam Safety
ECS – Existing Conditions Submission
ESRI – Environmental Systems Research Institute
ETL – Engineering Technical Letter
FDA - Flood Damage Assessment
FDD – Fairfax Drainage District
FOSM – First Order Second Moment
FS – Factor of Safety
GDS – Geospatial Data Services
GIS – Geographical Information System
GPS – Global Positioning System
GDT – Geographic Data Technology
HEC – Hydrologic Engineering Center
HGL – Hydraulic Grade Line
HQ – Head Quarters
HTRW – Hazardous, Toxic, and Radioactive Waste
KCD – Kansas City District
KCK – Kansas City, Kansas
KCMO – Kansas City, Missouri
KCPL – Kansas City Power & Light
KCS – Kansas City Southern
KCT – Kansas City Terminal
KVDD – Kaw Valley Drainage District
LE – Lower End
LERRD – Land, Easements, Rights-Of-Way, Relocation, and Disposal
ML – Silt
MoPac – Missouri Pacific Railroad

NAD – North American Datum
NAVD 88 – North American Vertical Datum of 1988
NED – National Economic Development
NGS – National Geodetic Survey
NGVD 29 – National Geodetic Vertical Datum of 1929
NKC – North Kansas City
NKCLD – North Kansas City Levee District
NOAA – National Oceanic & Atmospheric Administration
O&M – Operation and Maintenance
PDT – Project Design Team
PED – Preliminary Engineering & Design
PM – Project Manager
POF – Probability of Failure
PVC – Polyvinylchloride
R&U – Risk and Uncertainty
RCB – Reinforced Concrete Box
RCP – Reinforced Concrete Pipe
RM – River Mile
SM – Silty Gravel
SMF – Strength Mobilization Factor
SP – Poorly Graded Sand
UE – Upper End
UG – Unified Government of Wyandotte County
UGE – Underground Electric Line
UL – Utility Line
USACE – United States Army Corps of Engineers
USCS – Unified Soil Classification System
USGS – United States Geological Survey
UTM - Universal Transverse Mercator
VCP – Vitrified Clay Pipe
WWTP – Waste Water Treatment Plant

Kansas Citys, Missouri and Kansas
Flood Risk Management Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Final Feasibility Report

Chapter A-1

GENERAL

CHAPTER A-1
GENERAL

A-1.1 INTRODUCTION

The focus of the engineering effort during the feasibility study is on establishing project elements and features, developing design assumptions, technically evaluating alternatives, and collecting and assessing data. The engineering aspects of the study have been developed to the level of detail sufficient enough to prepare a baseline cost estimate, general project schedule, and allow for more detailed design of the selected plan following receipt of funds. The results of engineering investigations, studies, and designs are presented in this Engineering Appendix to the Final Feasibility Report.

A-1.2 NOTES REGARDING APPENDIX FORMAT

The development of the engineering appendix documentation progressed on two parallel paths, one for each unit: Armourdale and Central Industrial District. Thus, each unit generally has its own set of Engineering Appendix chapters. The Central Industrial District's (CID) two sections are usually broken out separately also, so there is a chapter for CID-KS and CID-MO, respectively, for each of the discipline-related analyses. The Civil Design and Pump Station Analysis chapters were developed slightly later than the others and the two CID sections were combined into one document each.

Please note the following:

- The overall method of chapter numbering was originally based on the Engineering Appendix to the Interim Feasibility Report (from Phase 1). However, some chapters from the previous report appendix did not apply to Phase 2 efforts, as they were specific to Phase 1. One chapter on bridge impact analysis was added specifically for Phase 2.
- Select report sections were not repeated as they apply to the entire feasibility report. Refer to the "Construction Procedures and Water Control Plan" and "Hydrology and Hydraulics" sections in the Engineering Appendix to the Interim Feasibility Report for information pertinent to those items.
- Each unit (or section) has a separate chapter for the same discipline, i.e. Chapter A-3 is "Geotechnical Analysis (Armourdale)", Chapter A-4 is "Geotechnical Analysis (CID-KS)", and Chapter A-5 is "Geotechnical Analysis (CID-MO)"
- The access roads discussion does not include the CID-MO as the feasibility study recommends only a small project that can utilize existing access.

A-1.3 BACKGROUND

The existing Kansas Citys Flood Risk Management Project provides local flood risk management for the metropolitan areas of Kansas City, Missouri and Kansas City, Kansas. The Kansas Citys project is a unit of the Missouri River basin comprehensive plan authorized by the 1936, 1944, 1946, and 1954 Flood Control Acts. A modification to raise some of the levee units (Argentine, Armourdale, Central Industrial District-Kansas, and Central Industrial District-Missouri) was authorized by public law in October, 1962. The design of the Kansas Citys project was predicated on the operation of the Kansas River Basin system of flood control lakes. Most of the lakes in that system are in place and operating, but two of the smaller lakes in the system (Grove and Onaga) were not economically feasible and have been deauthorized.

The study area consists of seven official levee units along reaches of the Kansas and Missouri Rivers. The Interim (Phase 1) Feasibility Report published in 2006 addressed recommendations for modifications to five of these units. The remaining two units, Armourdale and Central Industrial District (CID), are addressed in this Final (Phase 2) Feasibility Report.

The Armourdale and CID units work in concert with the Argentine Unit to create a three-unit system along the lower 10 miles of the Kansas River. The units were designed and constructed in conjunction with each other, but are independently operated to some extent. The total protected area is characterized by dense industrial and commercial development. Some limited residential habitation is also present. Communities (or portions thereof) within the study area include Kansas City, Missouri, and Kansas City, Kansas.

The U.S. Army Corps of Engineers Kansas City District, along with the local project sponsors, conducted a feasibility study of the existing flood protection project within the Kansas City metropolitan area. The study was authorized under Section 216 of the 1970 Flood Control Act (review of completed civil works). The entire metropolitan system of levee units withstood the Missouri River Flood of 1993, but some elements of the system were seriously challenged as the flood crest neared overtopping at some locations. This experience raised a concern that the levees may provide less than the level of protection for which they were designed.

The purpose of the feasibility study was twofold. First, it served to update and verify data on the reliability of the existing project (Kansas Citys, Missouri and Kansas, Local Flood Protection Project). Secondly, it provided a means to develop alternative plans (to include a review of the "no Federal action" alternative) with a final recommended plan for authorization and implementation. Any recommended plan for increasing the reliability of the system must be technically viable, economically feasible, and environmentally acceptable.

A-1.4 GENERAL DESCRIPTION OF LEVEE UNITS

A-1.4.1 Armourdale Unit

The Armourdale Unit is located along the left bank of the Kansas River from River Mile (RM) 6.4 to RM 0.3, near the confluence of the Kansas and Missouri River. The Kaw Valley Drainage District of Wyandotte County, Kansas, furnished the required assurances of local cooperation by resolution dated June 15, 1938. The original levees and floodwalls were constructed under the jurisdiction of the Kaw Valley Drainage District. The levee is separated into three sections totaling about 5.8 miles in length. The uppermost levee section originally was a tieback from high ground on the left bank of Mattoon Creek to the Union Pacific Railroad tracks. The levee was extended west past Mattoon Creek approximately 1,500 feet since its original design. From the Union Pacific Railroad tracks, the levee extends from the railroad embankment near the mouth of Mattoon Creek downstream along the left bank of the Kansas River to the floodwall. The second portion is a floodwall that begins north of the Chicago, Rock Island and Pacific Railroad Bridge and extends downstream to connect with the third levee section. The third levee section ties back into high ground at the embankment of the Lewis and Clark Viaduct.

Construction of the Federal project began in May, 1949 and was completed in February, 1951. More recent improvements, separately authorized as the 1962 Modification, were completed in April, 1976.

The flood protection unit consists of levees, stability berms, retaining walls, floodwalls, underseepage control including 45 relief wells, 2 sandbag gaps and 2 stoplog gaps, 10 pump plants, and 36 drainage structures. The levees stretch about 5.8 miles through the Armourdale Unit and the floodwalls total approximately 6,600 feet.

A-1.4.2 Central Industrial District – Kansas Unit

The Central Industrial District – Kansas flood protection unit is located in Wyandotte County, Kansas, and extends from the Kansas/Missouri state line along the right bank of the Missouri River to the mouth of the Kansas River. It then continues upstream along the right bank of the Kansas River to RM 3.4. The Kaw Valley Drainage District is the local agency responsible for operation and maintenance. The original unit was constructed by the Kaw Valley Drainage District prior to May, 1948, when initial improvements began. The bulk of the improvements were completed by November, 1955. The most recent improvements were completed in December, 1979.

The unit consists of a system of levees and floodwalls, underseepage control including 17 relief wells, a stoplog gap, a sandbag gap, 10 pump plants, and 23 drainage structures. The levee is approximately 1.8 miles long and the floodwalls total about 7,900 feet.

A-1.4.3 Central Industrial District – Missouri Unit

The Central Industrial District – Missouri flood protection unit is located in Kansas City, Missouri within Jackson County. The unit extends along the right bank of the Missouri River, upstream from the Grand Avenue Viaduct (Missouri RM 365.7), to the Kansas/Missouri state line (RM 367.2). The City Council passed four resolutions between 1941 and 1947 to provide the required assurances of local cooperation. The initial construction was in March, 1946 and construction was completed in September, 1947. Significant improvements and repair of 1951 flood damage followed the initial construction and were completed in November, 1955.

The unit consists of a system of levees, floodwalls, underseepage control, 1 sandbag and 7 stoplog gaps, 7 pump plants, and 5 conduits. The levees total about 430 feet in length and the floodwalls are about 1.45 miles long.

A-1.5 SPONSORS AND OWNERSHIP

Discussions with local sponsors have provided much of the information used in the Kansas Citys Flood Risk Management Feasibility Study. The local sponsors are listed below:

Unit	Sponsor
Armourdale	Kaw Valley Drainage District
Central Industrial District – Kansas	Kaw Valley Drainage District
Central Industrial District – Missouri	City of Kansas City, Missouri

A-1.6 PROJECT DESCRIPTION

A Corps of Engineers (COE) reconnaissance level report was completed in August, 1999. The effort included compiling a list of existing features and indicating the impact to those features due to a 1.5-foot and 3.0-foot levee raise for all units. The report indicated that raising the level of protection provided by the Kansas Citys system may be technically and economically feasible without unacceptable environmental or social impacts.

The Reconnaissance Report identified a Federal interest in further investigation of the drainage structures. That recommendation led to the current Feasibility Study. An early effort under feasibility was development of the Inventory of Drainage Features Report submitted to the COE and performed by HNTB Corporation in June, 2001. This inventory was published in the Engineering Appendix of the Interim Feasibility Report. The general purpose was to obtain original drainage designs of interior structures and to compare those designs with current conditions for each unit. More specifically, the tasks included the compilation of an inventory for each levee unit's drainage system capacity criteria and assumptions, along with the recording of flood protection penetration information for stormwater conduits.

The Inventory of Drainage Features Report was incorporated into work on existing conditions analysis of each unit in the protection system. Additionally, information was gathered (where available) from the original design documents, Operation and Maintenance (O&M) manuals, and associated studies. The Corps utilized current hydrology/hydraulics models, and geotechnical/structural risk and uncertainty (R&U) study methods to develop the engineering portions of the existing conditions (baseline) analysis of the existing project. Much of this analysis was based on data and observations from recent high water events (since the original project design), especially those in 1993 and 1995. This new engineering analysis, along with the economic (HEC-FDA) analysis, established a complete R&U approach to estimating existing conditions flood damages. The engineering and economic evaluations taken together with a summary baseline environmental review and an HTRW review of the study area formed the full picture of existing conditions. A review of existing conditions results by the study team provided guidance during the scoping and development of future conditions (with and without project) work.

Findings for overtopping risk and geotechnical/structural risk led the PDT to undertake evaluations in Phase 2 which were aimed at increasing the overall level of performance for the Armourdale and CID units. The resulting recommendations are for raises to each unit that would provide risk reduction at the 0.2% chance of exceedance (500-year) plus 3-foot water surface profile elevations along the protection. These raises and their associated improvements became the focus of the Engineering Appendix to the Final Feasibility Report.

In order to obtain a clearer overview of the specific areas of interest for Phase 2, please refer to the Maps section of the Final Feasibility Report. The footprint mapping details the location of proposed modifications and identifies some of the concerns and key issues to be addressed during design. Additionally, the Final Feasibility Report Exhibit #6 and Exhibit #7 present summary matrices detailing the recommended work at each location within the units. As subsequent chapters provide discussion of the areas of interest, these Maps and Exhibits will be valuable visual and summary references.

Kansas Citys, Missouri and Kansas
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Chapter A-2

SURVEYING, MAPPING, AND OTHER GEOSPATIAL DATA REQUIREMENTS

CHAPTER A-2
SURVEYING, MAPPING, AND OTHER GEOSPATIAL DATA REQUIREMENTS

A-2.1 GENERAL SURVEY AND DATUM INFORMATION

The need for datum consistency on the Kansas City's flood risk management project is essential. Since vertical reference datum uncertainties and deficiencies are known to exist within the Kansas City area and across the Missouri and Kansas state lines it is imperative that spatial datums be addressed during this feasibility stage and plans put in place to address these issues during detailed design.

During the time when the Kansas City's levee units were originally surveyed and constructed, benchmarks and survey control would have been set up locally around each individual unit isolated from other levee units. This independent approach of surveying and construction, although adequate at the time, is not in line with the "System's Approach" taken for the Kansas City's Flood Risk Management Project. Use of Global Positioning System (GPS) based survey equipment also allows for benchmarks and control to be located further from the individual units. For example, it is possible that a benchmark used for Argentine can easily be used for Armourdale. Because the Kansas City's levees act together during a flood event to protect the area, the entire system of levee units should be surveyed, such that there is no question how elevations from one unit relates to another.

The purpose of this chapter is to document the surveying, mapping and other geospatial data used or reviewed as part of the feasibility level design. From this review of data will come the recommendation to consider the entire seven levees when planning and performing survey and mapping activities for future detailed design phases. Survey data and mapping is the information base from which many engineering recommendations come and certainly the base from which detailed design and construction will begin.

A-2.1.1 Kansas City Area Survey Control

Within the Kansas City's Flood Risk Management project area, there are several systems of survey control in existence. These systems are maintained by various entities.

A-2.1.1.1 Kansas City, Missouri and Missouri Department of Natural Resources – Horizontal

The Missouri Department of Natural Resources directed the execution of a geodetic survey of part of the Kansas City, Missouri Metro Area. As part of this effort, it was able to show that all main stations were in conformity with the proposed Geometric Geodetic Survey Standards Version Four September 1, 1986 for First Order; and have been rigidly adjusted on NAD 1983 and whose coordinates have been computed on the Missouri Coordinate System of 1983 West Zone. This project also made direct ties between several NGS control stations within the area.

It is likely that this system of horizontal control will be incorporated into the survey plan for detailed design.

A-2.1.1.2 Kansas City, Missouri Directrix – Vertical

Kansas City, Missouri has an independent vertical datum plane called the Kansas City Directrix. According to an article on the KCMO Public Works website written by Sam Laffoon, former Chief of Surveys for the City of Kansas City, the Directrix was established about 1860 by the city engineer of that time. In 1892, a resolution by the council of Kansas City stated that the Kansas City Directrix was a plane 721.84 above mean sea level. Later records show that precise levels from Beloxi, Mississippi determined it to be 723.24 above sea level. After the general adjustment of 1929, the Geodetic Surveys changed their elevations in this area so that the equation was again about 721.24. Then in 1948 they announced the results of "releveling" that changed the equation to near 722.30, which is the conversion factor used today. This conversion factor applies only to National Geodetic Vertical Datum of 1929 (NGVD29) elevations.

It is likely that this system of vertical control will NOT be incorporated into the survey plan for detailed design.

A-2.1.1.3 Unified Government of Wyandotte County

The Unified Government use National Oceanic and Atmospheric Administration's (NOAA) National Geodetic Survey (NGS) system of points. This national system is discussed below.

A-2.1.1.4 US Army Corps of Engineers

The Corps of Engineers establishes survey control depending on the subject matter. The Corps has set up local control, used the NGS system, or other local controls already established. Harbor lines were established many years ago and monuments set for these may still exist.

It is likely that US Army Corps of Engineers monuments that can be found in the project area will be incorporated into the survey control system.

A-2.1.1.5 National Geodetic Study

The NGS defines and manages a national coordinate system. This network, the National Spatial Reference System (NSRS), provides the foundation for transportation and communication; mapping and charting; and a multitude of scientific and engineering applications.

NGS conducts aerial photography surveys near airports in the United States and its possessions to position obstructions and aids to air travel. NGS also maps the coastal regions of the United States and provides data for navigational charts. There are numerous NGS monuments throughout the Kansas City's Seven Levees Project area.

It is likely that this system of vertical control will be incorporated into the survey plan for detailed design.

A-2.1.2 Surveys

A-2.1.2.1 O&M Manuals and Record Drawings

The O&M Manuals and Record Drawings were reviewed and used extensively during the feasibility level design. A records search was performed to determine what survey information was available during the design and construction of the CID-KS Unit. Exhibit A-2.1 in the supplemental exhibits section of this chapter shows the benchmarks for the 1962 Modification of the Armourdale Unit.

A-2.1.2.2 2000 Surveys for Miscellaneous Feasibility Level Design Efforts

A centerline survey of the top of levee was conducted in April of 2001 for all seven units in the Kansas Citys protection system. The survey was conducted for verification of the O&M Manual elevations and used as a baseline for the hydrologic and hydraulic analyses. See Exhibit A-2.2, at the end of this chapter, showing the centerline survey elevations plotted along approximate levee stationing.

A review of the centerline survey indicated that some areas along the levee were lower than shown in the O&M Manual. Based on this, a resurvey of portions of the centerline was conducted in late 2003. The results of the resurvey confirmed that, in comparison with the original design elevations, several areas were lower. The following are the sections of levee units that were resurveyed:

- CID-MO Unit – Station 73+00 to Station 89+37
- CID-KS Unit – Station 0+00 to Station 3+00
- Fairfax-Jersey Creek Unit – Station 27+00 to Station 30+00
- Fairfax-Jersey Creek Unit – Station 303+00 to Station 295+00
- North Kansas City Unit – Station 90+00 to Station 110+00
- North Kansas City Unit – Station 265+00 to Station 285+00
- Armourdale Unit – Station 93+00 to Station 102+00
- Armourdale Unit – Station 246+00 to Station 252+00

In addition, the area between the CID-MO and East Bottoms Units was resurveyed to verify the intended line of protection (existing "high ground"). Contour mapping suggested that this area does not provide the same level of protection as the CID-MO or East Bottoms Units.

A-2.1.3 Geospatial Data

Around the beginning of Phase 1 of the feasibility-level design, the GIS members of the team were tasked with acquiring geospatial data from GIS vendors, local municipalities, etc. As this study area covers two different state plane zones, it was decided that all project geospatial data be in Universal Transverse Mercator (UTM) Zone 15, which covers the whole study area.

Geospatial data used for this project includes the following:

Acquired Data (see Exhibit A-2.3 for graphical summary at the end of this chapter)

- In 2001, the Unified Government of Wyandotte County provided two-dimensional survey data of the Argentine, Armourdale, and CID-KS Units. This topographic and planimetric AutoCAD mapping (103 files) has a projection of Kansas State Plane, North (feet), NAD 83. The elevations are based on Mean Sea Level, North American Vertical Datum (NAVD) of 1988. The 2D contours have a 2-foot interval. Concerning utility data, sanitary sewer, water, and electric AutoCAD data was acquired from three different points of contact within the Unified Government of Wyandotte County. Again, all of the utility data had a projection of Kansas State Plane, North (feet), NAD 83.

- The governments of Cass County, Missouri, the City of Kansas City, Missouri, and Wyandotte County, Kansas each provided land parcel spatial data and accompanying tabular data detailing appraised value, assessed value, land use, and ownership information.

- The project team purchased 3D data (ASCII point and break line data) that the contractor used to produce the Wyandotte County 2D contours. This 3D data was used to help identify the projected toe of the levee for the various proposed raise alternatives. This data had the same datum as the Wyandotte County information: Kansas State Plane, North (feet), NAD 83 and North American Vertical Datum (NAVD) of 1988.

Existing In-House (Corporate) Data

- The project team used Missouri River Microstation mapping. The Missouri River mapping was created in 1995 and 1998 and has a projection of UTM Zone 15 (feet), NAD 83. The topographic data, or 3D contours, have a 4-foot interval and a vertical datum of NGVD 29 feet.

- ESRI, GDT, and Navtech data sets were used as references. These USACE licensed, commercially derived, data sets are essentially census spatial data with more attribution and accuracy. These data sets were in a variety of projections and needed to be manipulated for use within the project.

- USGS's Digitally Ortho-rectified Quarter Quadrangles (DOQQ) black and white imagery was used throughout this project as import background and navigation information. The DOQQs have a 1-meter resolution. The dates of these DOQQs range from 1991 to 1997. The DOQQs are a part of the GIS team members' corporate data holdings, which were acquired through regional geospatial data clearinghouses.

- The USACE-KCD purchased commercially made digital ortho-rectified color imagery of the Kansas City metropolitan area. This imagery has a 2-foot resolution. The date of this imagery is June 2001.

Projects – Phase 1 and Phase 2 Feasibility Level Design

- Data Acquisition – October 2000 through March 2001 – Phase 1
  - Much of this time was used to data mine for Kansas River spatial data and arrange for acquisition of this data

- Data Processing – Phase 1 & 2
  - All of the Wyandotte County AutoCAD data was converted to Microstation format and then projected from Kansas State Plane, North (feet), NAD 83 to UTM Zone 15 (feet), NAD 83

- HEC-FDA Model Inputs (Economic and H&H support) – October 2001 through March 2003 – Phase 1
  - The GIS members of the project team were tasked with mapping support for the economics field inventory efforts. After acquiring the land parcel spatial and tabular data from the sources listed above, this land parcel data and their parcel numbers were displayed over the top of contour, building, and road name spatial data with the DOQQ's as a background. These maps helped the USACE economists acquire information about possible damage assessment associated with levee failure. The USACE economists used these maps to complete field studies to gather new, accurate data about building wealth in the Kansas City metro area.

  - The GIS members of the project team were also tasked with mapping support for the H&H efforts. H&H team members requested a set of comparison maps showing the relationship between river miles and existing levee structures where they cross the top of levee centerline. This data later aided in comparing the top of levee elevation to the water surface elevation of a 0.2% chance of exceedance (500-yr) flood event.

- Levee Raise Layouts (existing & new top of levee) – Phase 1 & 2
  - The GIS members of the project team were tasked with creating a map set showing the location of existing levee structures where they cross the top of levee centerline. Color-coded station number text was included to coincide with the Levee/Floodwall Features Inventory spreadsheet information and to help categorize each feature's point. The project team needed this mapping for eventual use in this engineering appendix document.

  - The GIS members of the project team were also tasked with creating a map set showing the location of specific zones of inundation near selected pump stations. Again, the project team needed this mapping for eventual inclusion in this engineering appendix.

- Lastly, the GIS members of the project team were tasked with creating a map set showing the footprints of potentially affected areas, borrow areas, and utility line (UL) uplift concern areas. Also needed was the location of existing levee structures where they cross the top of levee centerline. The project team utilized this mapping to analyze areas of concern.

- Feature Inventory Delineation Maps – Phase 1 & 2
  - The GIS members of the project team were tasked with adding onto the information created with the existing condition maps by adding top-of-levee elevation and description text to the existing levee station text. By adding this information to the map, map users are able to gather most of the aforementioned spreadsheet information without referring to the spreadsheets.

- Utility Site Maps – Phase 1 & 2
  - For the Argentine Unit, the GIS Section created a map set showing where utilities cross the levee. At these crossings, there are specific text boxes giving data about the crossings.

  - Hard copies of gas maps were obtained from Kansas Gas Energy to supplement electronic information obtained. Both hard copy and electronic data was referred to when evaluating impacts to utilities.

- Real Estate Support – Phase 1 & 2
  - The GIS members of the project team were tasked with creating map plates similar to those created for the economic field survey discussed above, but without the contour information. Real estate personnel on the project team needed this information for their analysis.

  - For the Argentine Unit, the real estate team members needed a group of maps showing the three alternatives being considered: raising the levee to a 500 year level of protection, raising it to 500-year plus 3-ft, and raising it to 500-year plus 5-ft. Each of these alternatives has a different set of files that include stability berms, proposed levee raise, proposed I-walls, proposed floodwalls, temporary right of ways, and outside COE property areas that the COE may have to purchase. This map set was generated fora sponsor meeting on 28 January 2004.

- Feature Inventory Delineation Maps – Phase 1 & 2
  - GIS members were tasked with similar activities during Phase 1 and Phase 2. They created feature delineation maps for the Armourdale Unit consisting of Existing Levee Stationing, Existing Levee Features, Recommended N500+3 raises and features, Utility crossings, and Real Estate Parcels and right of way.

A-2.1.4 Datum Relationship

Table A-2.1 "Survey Datums and Benchmarks" shows the sources of survey and mapping data that was used during the CID-KS feasibility level design. During Phase 1 of the feasibility design, the PDT determined that a common datum was needed when referencing various sources of data. The horizontal datum would be UTM Zone 15, NAD 83 and the vertical datum would be NGVD 29. When data was obtained in a datum other than that desired, it was converted before using. The included table shows that all sources of data used during feasibility had the same datum. An attempt was made to determine if any of the surveys conducted used any of the same benchmarks or monuments. It was desirable to perform a check between any of the sources of data. It was unable to be determined if any of the surveys used any of the same benchmarks or monuments. However, because a consistent horizontal and vertical datum was used, the data should be relatively compatible for feasibility purposes with no known discrepancies.

TABLE A-2.1
Survey Datums and Benchmarks

Data Title and Source	Date Surveyed	Horizontal Datum		Vertical Datum		Comments	Used during Phase 2 Feasibility by:
		Original	Converted to and Used	Original	Converted to and Used		
Operation and Maintenance Manual, Record Drawing, O&MM Plate No. 164: Location and Vicinity	1972			NGVD 29 feet	No conversion necessary	Survey by Benton	Geotech, Civil Design
Levee Centerline Survey by KC Corps	Apr-2001 and 2003	KS State Plane	UTM Zone 15 (feet), NAD 83	NAVD 88 feet	NGVD 29 feet		H&H model
Unified Government of Wyandotte County (2-D data)	Various (1989-2000)	KS State Plane North (feet), NAD 83	UTM Zone 15 (feet), NAD 83	NGVD 29 feet	No conversion necessary	2-foot contour interval	Geotech
Unified Government of Wyandotte County (3-D data)	Various (1989-2000)	KS State Plane North (feet), NAD 83	UTM Zone 15 (feet), NAD 83	NGVD 29 feet	No conversion necessary	2-foot contour interval. Obtained from MJ Harden	Geotech
MO River Microstation Mapping (Aerial survey)	1995, 1998	UTM Zone 15 (feet), NAD 83	No conversion necessary	NGVD 29 feet	No conversion necessary	4-foot contour interval	H&H model
Digital Orthorectified Color Imagery	Jun-2001						Feasibility Phase 2 Map Book
Hydrographic Survey by Napoleon Crew	1998 and 1999	UTM Zone 15 (feet), NAD 83	No conversion necessary	NGVD 29 feet	No conversion necessary		H&H model

Table Created: March 5, 2007

A-2.2 SURVEY INFORMATION NEEDED FOR DESIGN, PLANS, AND SPECIFICATIONS

Survey information used during feasibility was from a combination of sources of varying degrees of age and accuracy. While this data is acceptable for feasibility, it is inadequate for design and construction. A completely new survey will be conducted prior to developing plans and specifications for construction. The development of plans and specifications requires a higher degree of accuracy and detail in regards to existing elevations, structures, utilities, and other items. Projects at the following levee units have been preliminarily recommended as a result of the Kansas City Levees feasibility level design:

- North Kansas City – Harlem, National Starch
- Fairfax - BPU
- East Bottoms – Blue River Confluence
- Argentine – Full Levee Raises
- Armourdale – Full Levee Raise
- CID – Full Levee Raise

According to Engineering and Design Interim Guidance for a Preliminary Evaluation of Vertical Datums on Flood Control, Shore Protection, Hurricane Protection and Navigation Projects, dated October 31, 2006, projects that are defined by a superseded datum such as NGVD 29, are in need of updating. Since the entire Kansas Citys system was designed and constructed in NGVD 29, it is recommended that detailed survey efforts necessary to conduct detailed design (PED) be done in NAVD 88. This recommendation acknowledges that risk exists with interchanging data with different datum. Designers will need to know and understand from where their data originates. While vertical control will be NAVD 88, horizontal control will be UTM Zone 15 (feet), NAD 83.

Based on the data reviewed and the directives recently initiated by USACE Headquarters, it is recommended that all surveys completed for any of the Kansas Citys levee units, be coordinated and looped together to establish reliability and consistency among the system. The existing control systems in existence around the area should prove sufficient to complete the loops. However, because there are known deficiencies between various control systems, the survey task will not be straight forward but require research and care in selecting monuments. An overall loop encompassing all of the seven levee units may not be feasible under any one approved project, thus a plan to loop the entire system will be created consisting of several interconnected subloops. See Exhibit A-2.4 at the end of this chapter depicting this concept. It is anticipated that surveys will incorporate state of the art GPS technology allowing for easier connectivity between units.

A-2.3 REFERENCES

1. US Army Corps of Engineers (January 1, 2007), *EM 1110-1-1005 CECW-CE Control and Topographic Surveying*, Kansas City District.
2. US Army Corps of Engineers (October 31, 2006), Engineering and Design Interim Guidance for a Preliminary Evaluation of Vertical Datums on Flood Control, Shore Protection, Hurricane Protection, and Navigation Projects

A-2.4 SUPPLEMENTAL EXHIBITS

EXHIBIT A-2.1
1962 Modification Armourdale Unit Benchmarks

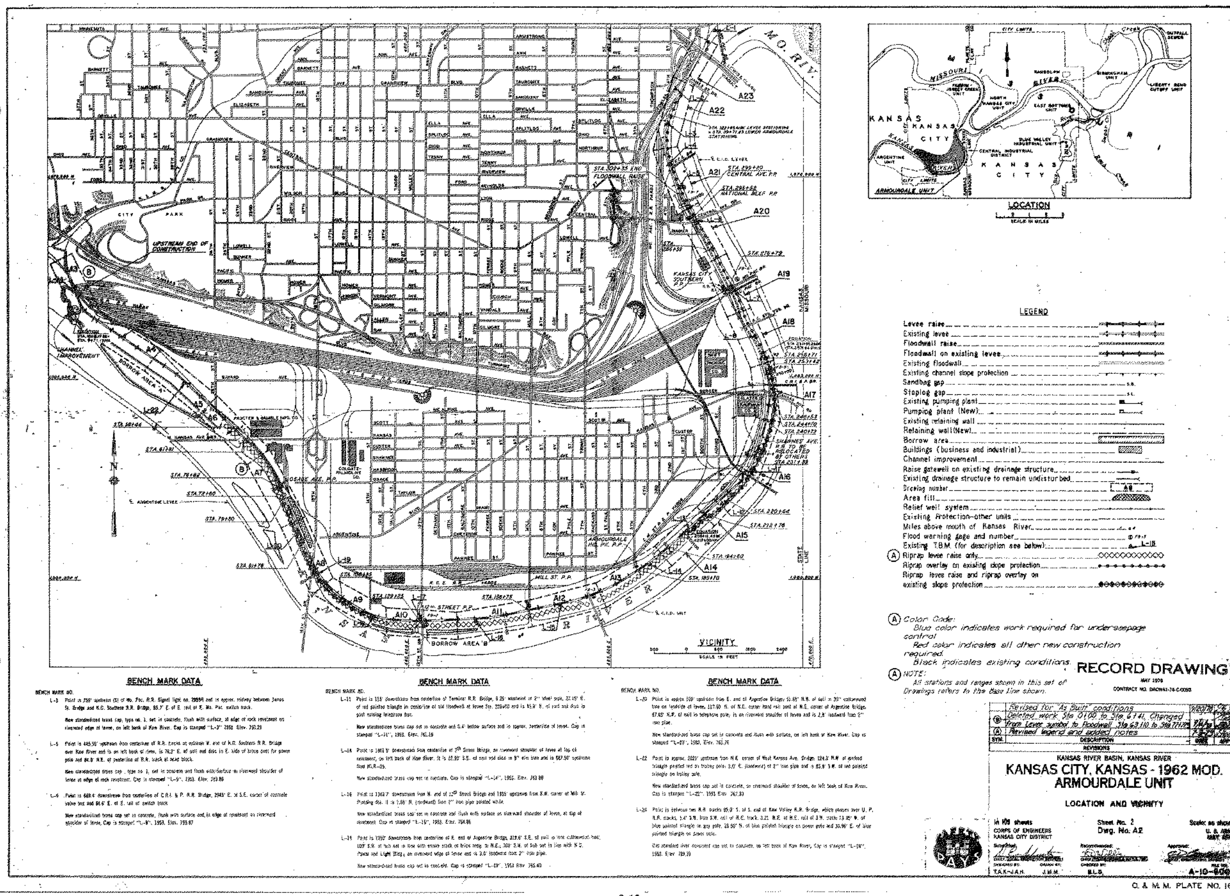


EXHIBIT A-2.2
Centerline Survey Elevations Along Approximate Levee Stationing

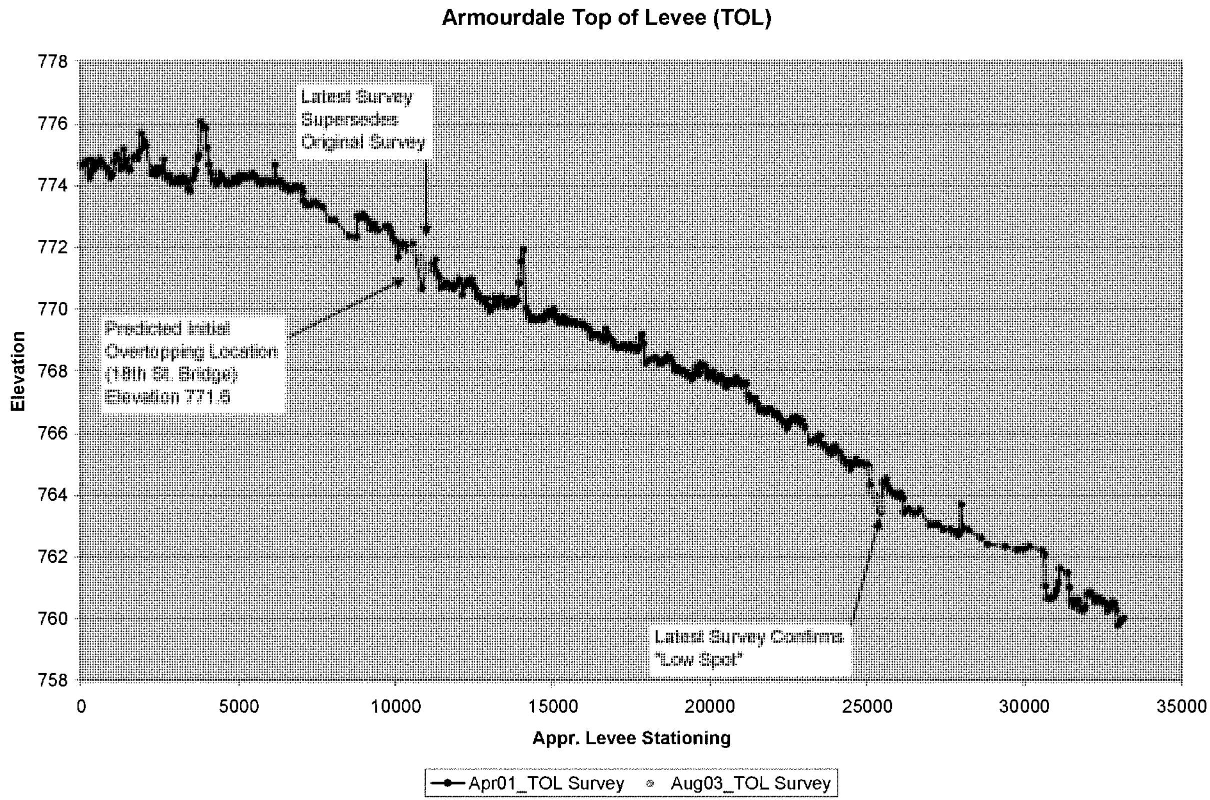


EXHIBIT A-2.3

Kansas Citys, Missouri and Kansas, Flood Risk Mauagement Report Feasibility Study Currency of Acquired Data

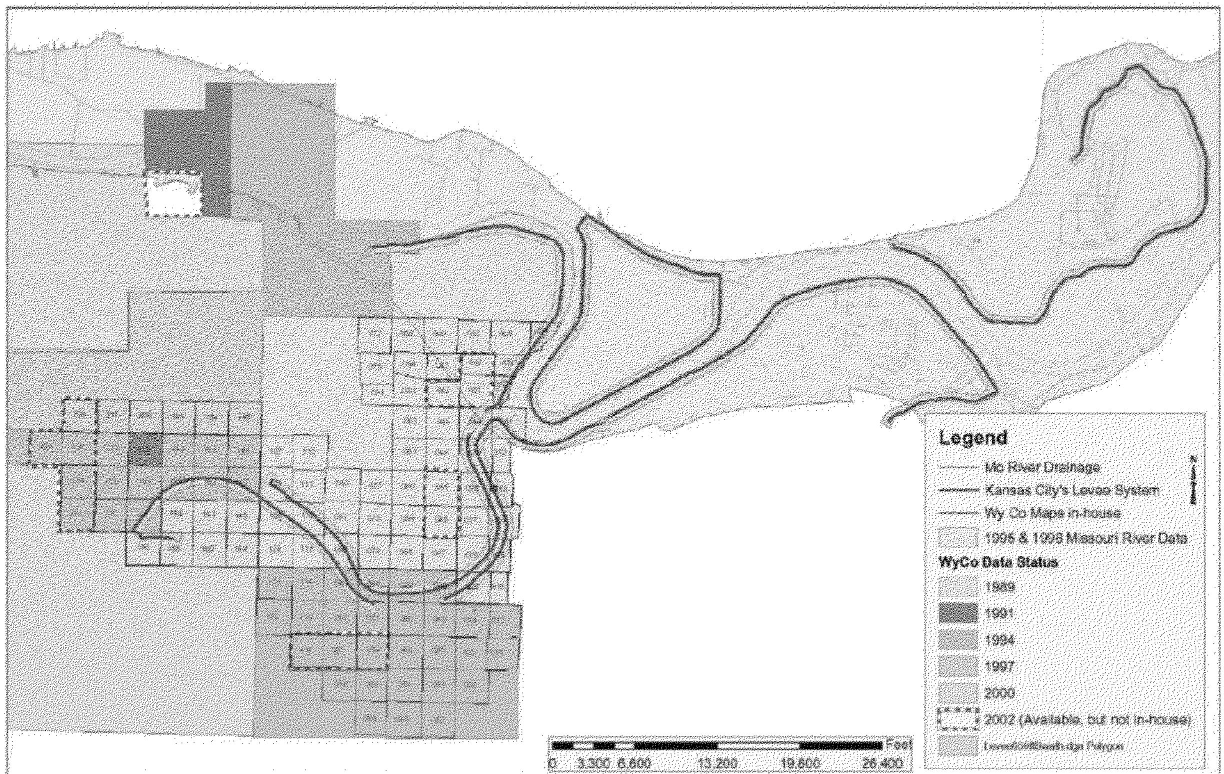
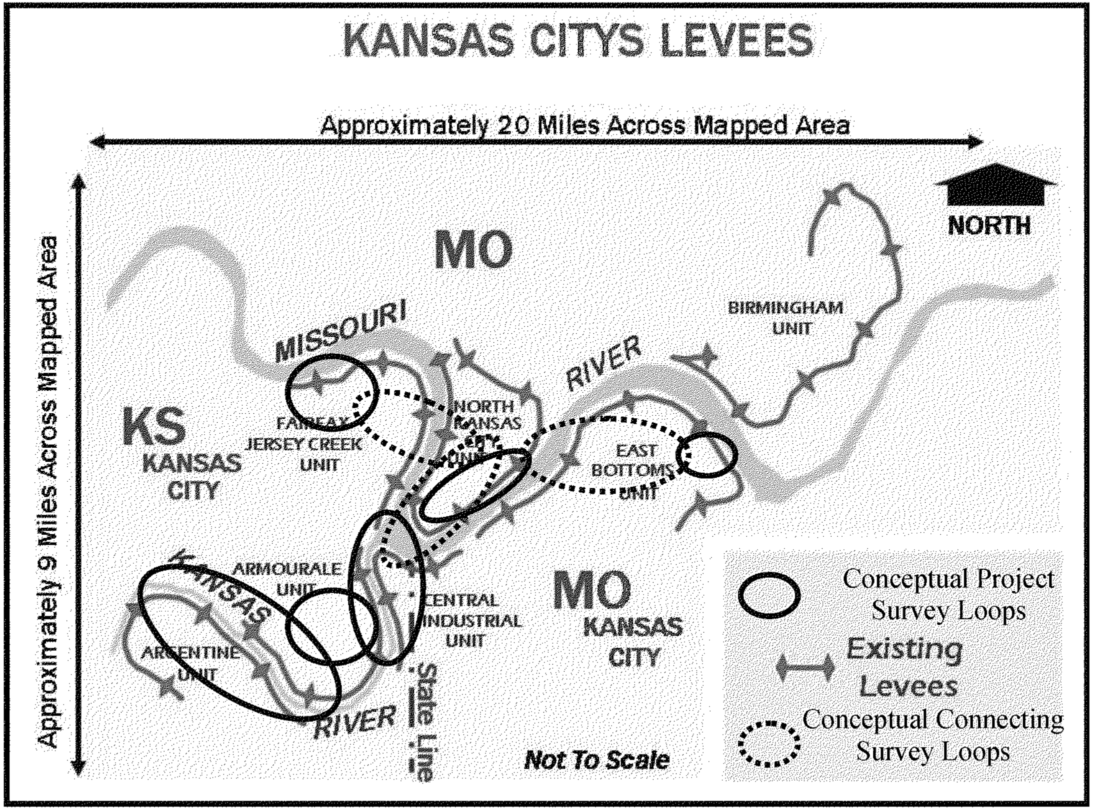


EXHIBIT A-2.4
Conceptnal Land Survey Loops



Kansas Citys, Missouri and Kansas
Flood Risk Management Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Final Feasibility Report

Chapter A-3

GEOTECHNICAL ANALYSIS ARMOURDALE

CHAPTER A-3
GEOTECHNICAL ANALYSIS - ARMOURDALE

A-3.1 INTRODUCTION

This chapter of the engineering appendix presents the results of the geotechnical evaluation performed for the Armourdale Unit in the Kaw Valley Drainage District. The evaluation started with a thorough review of existing project documentation, definition of existing subsurface conditions along the entire unit based upon existing subsurface information, and estimation of soil parameters for the existing levees, the natural blanket, and the aquifer materials. The estimated soil parameters are based on geotechnical laboratory testing data from Design Memorandum No. 3. All elevations used in the geotechnical portion of the feasibility study are NGVD 29.

Geotechnical analysis of the unit consisted mainly of underseepage and stability calculations for the following loading conditions:

- Existing Conditions, to identify the most critical areas with respect to risk of failure for use in the HEC-FDA economic model.
- Proposed Design Conditions, all of which included raising the current level of protection as follows:
    - Nominal 500 year flood event (N500+0), i.e. a 0.2% chance of occurrence in any one year
    - Nominal 500 year flood event plus 3-ft (N500+3)
    - Nominal 500 year flood event plus 5-ft (N500+5)

The majority of the design work focused on the N500+3 flood event. The raise above the current level of protection varied from 3.8-ft to 5.2-ft, except for a short section near the confluence with the Missouri River where the raise varied approximately from about 1.2-ft to 1.8-ft.

Underseepage was addressed along the entire Armourdale Unit. Where calculations showed that hydraulic gradients in the natural blanket did not meet current criteria for the N500+3 design condition, seepage control measures were designed to reduce the gradient in order to meet criteria.

Slope stability analyses were performed for three different levee sections:

1. An earthen levee raise with fill on the riverside of the protection
2. An earthen levee raise with fill on the landside of the protection
3. A cantilever retaining wall raise on top of an existing levee

An attempt was made to identify the most critical sections for each type of raise for the analyses. The sections were selected based on the initial height of the protection, the amount of raise proposed, and the pore pressures in the natural blanket based upon the underseepage calculations. Each of the sections analyzed were modified as necessary to obtain the required factor of safety against sliding for both the end of construction case and the steady seepage case. Rapid drawdown was not considered due to the lack of existing strength data required for this analysis.

The results of the analyses are discussed in additional detail in later sections of this chapter. For the purposes of economic modeling of proposed levee raises, all features which meet current Corps of Engineers criteria are arbitrarily assigned a reliability of 99.8%.

A-3.2 DESCRIPTION OF EXISTING LEVEE UNIT

A-3.2.1 Levee Description

The Armourdale Unit is located in Wyandotte County, Kansas on the left bank of the Kansas River between approximate Kansas River miles 7.0 and 0.25. Mile 0.0 is at the confluence with the Missouri River. The levee begins at Station 0+00 UE (Upper End) where it ties into high ground across the Kansas City Southern Railroad tracks just downstream of the I-635 bridge, and extends downstream to Station 61+00 LE (Lower End). The unit protects numerous commercial and light industrial properties as well as a significant residential population. The total length of the unit is 34,853.45 feet or about 6.6 miles.

The line of protection alignment stationing has been separated into three designations due to several changes made to the alignment throughout the history of the unit. The upper end stationing begins at 0+00 UE and ends at 20+08.89 UE BK (back). A station equation is inserted at this point to change the station to 9+71.16 AH (ahead). The stationing continues to 206+12.43 BK. This location has a station equation that changes to 212+00 AH. The stationing then remains consistent to 257+66.26 BK, where another station equation is inserted to change the station to 257+64.97 AH. The stationing continues to 322+85 BK. At this point a final station equation is inserted which defines the lower end and changes the stationing to 39+71.83 LE. The unit finally ends at station 61+00 LE.

There are many bridges, structures, and utilities within the critical area of the line of protection. For the purposes of the feasibility study, it was assumed that all bridge foundation elements, structures, and utilities within the levee embankment and critical area of the foundation blanket material meet all pertinent Corps of Engineers criteria.

A-3.2.2 History

The Kaw Valley Drainage District initiated work on the Armourdale Unit prior to any involvement by the Federal Government. Previous works included the construction of earthen levee sections, drainage structures, and even pump plants. The Flood Control Act of 1936 authorized the Corps of Engineers to provide assistance. Work began to improve parts of the project in 1949. The flood of 1951 caused extensive damage to the original levee, and the Corps of Engineers designed and constructed the restoration of the protection. The Corps of Engineers again became involved in the 1960s to raise the level of protection along the Armourdale Unit - reference Design Memorandum No. 3. The raise was constructed in the early 1970s. The following discussion describes the unit in additional detail by major features.

Station 0+00 UE to 2+90 UE
This is a stoplog gap across several sets of railroad tracks, and starts the upper end of the project. Two sets of openings exist across this significant span. This work was part of an upper end extension that was a modification to the original 1962 Modification as outlined in Design Memorandum No. 3.

Station 2+90 UE to 60+40
This section of the line of protection is an earth fill levee section.

Station 60+40 to 77+77.5
This section is a floodwall that extends across the West Kansas Avenue Bridge abutment. The bridge has since been rebuilt and the original stoplog section at Station 62+30 has been replaced with floodwall.

Station 77+77.5 to 226+01
This section of the line of protection is an earth fill levee section. A short retaining wall exists along the landside toe starting at approximately Station 212+00 and continues to 226+10. The retaining wall was constructed to avoid fill placement on an existing railroad track that parallels the levee. The railroad subsequently has been abandoned.

Station 226+01 to 227+46
This is a floodwall and stoplog gap section for the Kansas City Terminal Bridge.

Station 227+46 to 246+88
This section of the line of protection is an earth fill levee section. This reach of the line of protection also has a landside retaining wall and parallel railroad track.

Station 246+88 to 250+52
This is a floodwall and stoplog gap section for the East Kansas Avenue Bridge.

Station 250+52 to 257+66.26 BK
This section of the line of protection is an earth fill levee section. This reach of the line of protection also has a landside retaining wall and parallel railroad track.

Station 257+64.97 AH to 302+57.65
This section of the line of protection is a floodwall. The wall ties into high ground at a railroad embankment that follows the bluff of the left river bank. The floodwall has stoplog or sandbag gaps for the Missouri Pacific and Union Pacific railroad bridges, as well as for the Central Avenue Bridge.

Station 302+57.65 to 61+00 LE
This section is a railroad embankment located at the toe of the bluff of the left river bank.

Based upon the record drawings, the existing levee sections have a thick impervious riverside section and a random fill section on the landside. For the upper reaches of the project from approximately Station 205+00 to 257+00, there is a pervious fill section protected with riprap on the riverside slope.

A-3.2.3 General Geology of the Region (Kansas River)

The Kansas River Valley, near its mouth, is cut into Pennsylvanian bedrock of the Missourian Series. The oldest bedrock exposed is the Bethany Falls Limestone member of the Swope Limestone formation, Kansas City Group. Bedrock of the Missourian Series is characterized by numerous limestone beds separated by clayey to somewhat sandy shale. The bedrock is generally overlain by much younger unconsolidated materials consisting of glacial drift, loess of the Pleistocene age, alluvium deposits and isolated remnants of till of Kansas stage ice sheet occurring on the hilltops. The Kansas River is near the southern edge of Kansas glaciation. Wind blown deposits of silt (loess) form an irregular deposit covering much of the eastern part of Wyandotte County. Alluvium, ranging from clay and silt to sand and gravel, occurs in the Kansas River Valley. Much of this alluvium is probably of glacial origin, having been deposited as glacial outwash from the melting ice sheets.

A-3.2.4 Subsurface Conditions

Assessments of the subsurface conditions for the Armourdale project were derived from the Record Drawings, Design Memorandums and borings made at selected sites during the feasibility study. Typical subsurface blanket conditions for Station 0+00 UE to Station 190+00 generally consist of silts, sandy clays and lean clays of average thickness ranging from 13-ft to 40-ft. Beyond Station 190+00 to the Lower End of Armourdale, the foundation blanket has multiple layers of sand intermixed with clays and silts. The aquifer thickness ranges from 25-ft to 77-ft. Groundwater levels are dependent on the seasonal changes and rises in the river. The subsurface investigation measured the water levels in the borings after allowing disturbances due to drilling to stabilize. The water levels are shown on drill logs and recorded on the strip log summary. In general the water levels measured adjacent to the existing level of protection were on average at least 15-ft below the landside ground surface for normal river levels.

A-3.2.5 Existing Underseepage Control Features

Throughout the existence of the Armourdale Levee Unit, many underseepage control measures have been constructed to aid in the prevention of developing an underseepage condition that could cause a levee failure. Underseepage control measures were designed and constructed during the restoration of the levee unit after the 1951 Flood, and during the 1962 Modification of the unit.

The underseepage control feature designed and constructed during the restoration after the 1951 Flood was an extensive impervious fill on the riverside of the levee to prevent seepage through sand lenses in the stratified natural blanket. The impervious fill "cutoffs" are extensions of the impervious fill section in the levee embankment. The impervious fill cutoff is a minimum of 5-ft in thickness measured normal to the slope. The impervious fill is protected from erosion and scour by stone riprap protection. The impervious fill was extended to varying elevations sufficient to ensure full cut off of sand lenses in the natural blanket. Typical cross sections, revised for the "As-Built" conditions can be found in Armourdale Unit Record Drawings Volume 1, O&M Plate No. 75-78, 126-128, 149-150, and 101-102. The extents of the riverside impervious fills are summarized in Table A-3.1:

Table A-3.1
Extents of the Riverside Impervious Fills

Beginning Station	Ending Station	Low Elevation of Riverside Impervious Cutoff
65+00	69+00	740
69+00	85+00	735
85+00	90+75	732
90+75	131+00	735
131+00	193+50	740
193+50	195+00	735
195+00	199+00	740

Underseepage control features designed and constructed during the 1962 Modification are detailed below. Additional details can be found in Design Memorandum No. 3.

<u>Station 78+50 to 94+00</u>
An aerial fill was constructed in a low lying area landward of the levee. The fill was designed and constructed as an underseepage berm. The aerial fill was constructed to elevation 760.0 and extends up to 200-ft landward of the levee centerline. The downstream limit of the aerial fill tied into the 18th Street roadway embankment. The aerial fill was designed to provide a factor of safety with respect to hydraulic gradient of 1.5 at the landside levee toe, and 1.1 at the berm toe, with the water at approximately 3-ft below the levee crest. Details on the aerial fill can be found in the Armourdale Unit Record Drawings Volume 2, O&M Plate No. 169-170.

<u>Station 190+00 to 248+00</u>
A relief well system, consisting of 24 fully penetrating artesian relief wells, was installed to remediate a series of underseepage concerns mostly related to existing building foundations. The relief well system was designed to provide a factor of safety with respect to hydraulic gradient of 1.5 at all check points, and 1.0 in basements or pits, with the water at the top of the levee. The wells are variably spaced and connected by a gravity header system which discharges into the Shawnee Avenue Pump Station. The relief well header system and pump station were designed to handle a maximum flow from the relief well system of 32 cfs, with the flow from each well assumed to be 1.33 cfs. An aerial fill was constructed in a low lying area between Stations 220+00 and 226+50 to supplement the relief well system. The aerial fill was constructed to elevation 749.0 and extends up to 300-ft landward of the levee centerline. Details on the relief well system can be found in the Armourdale Unit Record Drawings Volume 2, O&M Plate No. 175-179, 197-200, 201, and 204.

<u>Station 274+00 to 283+00</u>
A relief well system, consisting of 8 fully penetrating artesian relief wells, was installed to protect a low lying railroad bed (which is now abandoned) directly adjacent to the landside of the existing floodwall. This area is commonly referred to as the "slot" area, and has an elevation up to approximately 15-ft below the surrounding ground. The system originally consisted of 14 wells, but portions of the old railroad bed have been filled and 6 of the wells have since been abandoned. The relief wells discharge into manholes that discharge through lateral pipes directly into the "slot" area. The relief wells serve two purposes:

1. Pressure relief at the base of the blanket
2. The discharge ponds in the slot and further reduce the gradient through the blanket

The relief well system was designed to provide a factor of safety of 1.0 with 5-ft of water ponded in the slot. The well flows were assumed to be between 1.75 and 2.0 cfs. Details on the relief well system can be found in the Armourdale Unit Record Drawings Volume 2, O&M Plate No. 180-181, 197-200, 205.

<u>Station 295+00 to 305+00</u>
A relief well system, consisting of 7 fully penetrating artesian relief wells, was installed to protect a large low lying area which contained a packing plant (which is no longer present) approximately 100-ft from the landside toe of the floodwall. The relief well system was designed to provide a factor of safety with respect to hydraulic gradient of 1.5 at all check points, and 1.0 in the basement of the packing plant, with the water at the top of the levee. The wells are connected by a gravity header system which discharges into the Central Avenue Pump Station. The relief well header system and pump station were designed to handle a maximum flow from the relief well system of 10.5 cfs, with the flow from each well assumed to be 1.5 cfs. Details on the relief well system can be found in the Armourdale Unit Record Drawings Volume 2, O&M Plate No. 182-183, 197-200, 205.

A-3.2.6 Overall Underseepage

For the underseepage analysis, the entire Armourdale Unit was divided into reaches of similar protection height, blanket thickness, blanket composition, aquifer thickness, and seepage entrance conditions. The factor of safety with respect to hydraulic gradient through the natural blanket was calculated for each of these reaches at the landside toe of the levee section or floodwall. Exhibit A-3.1, located at the end of this chapter, shows the calculated factor of safety with respect to hydraulic gradient for the entire Armourdale Levee Unit (without the effects of existing relief wells or cutoff walls), as well as the parameters used to calculate the factor of safety with respect to hydraulic gradient.

A-3.3 SOIL STRENGTH PARAMETERS

The required parameters for soils in the Armourdale Unit reach were estimated mainly from the significant amount of geotechnical laboratory testing performed for the 1962

Modification and provided in Design Memorandum No. 3. A summary of the soil parameters is provided in Table A-3.2 below and discussed in the following paragraphs.

Table A-3.2
Geotechnical Design Parameters

Material	Unit Weight		Shear Strength			
	Moist	Saturated	Undrained		Drained	
	g (pcf)	g (pcf)	c (psf)	f (deg)	c' (psf)	f' (deg)
Levee Fill	115	120	1000	0	0	29
Foundation Blanket	110	115	500	0	0	26
Foundation Sands	115	120	N/A	N/A	0	32

The existing levee sections consist of a riverward impervious zone and landward random fill zone, and toward the lower end of the unit there is also a pervious fill section on the riverside. To simplify the analyses, one set of parameters was used for the entire levee section and was called "levee fill".

The blanket materials consist mostly of ML and CL materials, with some discontinuous layers of CH and SM material. Design Memorandum No. 3 presented the laboratory test results sorted by soil classification. To simplify the analysis for this study, the blanket was modeled as a single material with only one set of strength parameters used. The soil strength applied to the blanket was a weighted average of the strength parameters for CL, ML and CH from the Design Memorandum.

Undrained shear strength data was not readily available for most of the materials, so undrained strengths were estimated from the limited 1962 Modification test data and typical values for these types of soils. Foundation blanket strength data was increased slightly from the existing test data to account for an increase in material strength under the footprint of the existing levee due to consolidation from the weight of the levee. It is recommended that additional sampling and testing be performed during PED to verify the undrained strength of the blanket materials.

A-3.4 EXISTING CONDITIONS RELIABILITY ANALYSIS

A-3.4.1 Introduction

The purpose of this portion of the study was to determine the probability of failure of the Armourdale Levee Unit for the existing condition of the unit. The analysis considered both underseepage piping failures and landward slope failures under steady state seepage conditions. The evaluations were performed in general accordance with the USACE Engineering Technical Letter (ETL) 1110-2-556 "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies." The results of the analyses were used to determine the economic benefits attributed to proposed levee raises.

A-3.4.2 Probabilistic Theory

A-3.4.2.1 Probabilistic Parameters

Several parameters are commonly used to describe probability distributions such as the normal distribution shown in Exhibit A-3.2 at the end of this chapter. Probably the most common of these is the mean or expected value. The expected value of a continuous random variable X (a variable that can take on any value within some continuous range) with some distribution $f(x)$ is defined as:

$$\mu_X = \int_{-\infty}^{\infty} x_i f_X(x) \; dx \qquad \text{Equation A-3.1}$$

where μ_X is the mean value of the random variable X, x_i is a particular value of the random variable X and $f_X(x)$ is the frequency of occurrence of the random variable X. The expected value, or mean, of a random variable is the weighted average of the values of the random variable with the weighting being the frequency of occurrence of the value. For a set of discrete measurements of a random variable, the mean value is computed as:

$$\mu_X = \frac{\sum_{i=1}^{N} x_i}{N} \qquad \text{Equation A-3.2}$$

The variance of the random variable X, Var[X], is a measure of the spread, or variability of the random variable about the mean. The variance is computed as:

$$\text{Var[X]} = \int_{-\infty}^{\infty} (x_i - \mu_X)^2 f_X(x) \, dx \qquad \text{Equation A-3.3}$$

For a set of discrete measurements of a random variable X, the variance is computed as:

$$\text{Var}[X] = \frac{\sum_{i=1}^{N} (x_i - \mu_X)^2}{N} \qquad \text{Equation A-3.4}$$

If the number of observations N is a relatively small set of an entire population, an unbiased estimate of the variance can be given as:

$$\text{Var}[X] = \sigma_X^2 = \frac{\sum_{i=1}^{N} (x_i - \mu_X)^2}{N-1} \qquad \text{Equation A-3.5}$$

The standard deviation, σ_x, is also a measure of the distribution of the random variable about the expected value and is the square root of the variance:

$$\sigma_X = \sqrt{Var[X]} \qquad \text{Equation A-3.6}$$

The coefficient of variation, COV, is a convenient dimensionless parameter used to express the uncertainty or variability of a random variable and is computed as:

$$\text{COV} = \frac{\sigma_X}{\mu_X} \qquad \text{Equation A-3.7}$$

The coefficient of variation is useful because it expresses the variability of a random variable normalized with respect to the mean of the random variable. The expected value, standard deviation and coefficient of variation are interrelated; therefore, the third can be determined by knowing any two of the parameters.

A-3.4.2.2 Probability Distributions

Many forms of probability distribution are available that can be used to represent the variability and uncertainty. However, based on previous work (Kitch, 1994) the normal and log-normal distributions are by far the most commonly used for risk based analyses.

The normal distribution is the most widely used distribution in the description of statistical phenomenon. The probability density function for a normally distributed random variable is expressed as:

$$f_X(x) = \frac{1}{\sigma_X\sqrt{2\pi}}\exp\left[-\frac{1}{2}\left(\frac{x-\mu_X}{\sigma_X}\right)^2\right]dx \qquad \text{Equation A-3.8}$$

where $f_X(x)$ is the relative frequency of the random variable X and is not a probability, but a representation of the distribution of probability that a particular random variable may lie within some stated interval. As shown in Exhibit A-3.2 the normal distribution has a bell shape with upper and lower limits of positive and negative infinity.

Another distribution that has been proven useful for reliability-based analysis in geotechnical engineering is the log-normal distribution shown in Exhibit A-3.3 at the end of this chapter. In the log-normal distribution, it is assumed that the natural logarithm of a random variable X is normally distributed. As shown in Exhibit A-3.3, the log-normal distribution is positively skewed towards the lower values. However, it has the distinct advantage that the probability of the random variable cannot be less than zero. The log-normal distribution is therefore useful for representing parameters that cannot take on negative values (e.g. factors of safety and hydraulic gradient).

If a random variable X is log-normally distributed, the *ln X* is normally distributed. The probability density function can therefore be expressed as:

$$f_X(x) = \frac{1}{x\sigma_{\ln X}\sqrt{2\pi}}\exp\left[-\frac{1}{2}\left(\frac{\ln x - E[\ln X]}{\sigma_{\ln X}}\right)^2\right]dx \qquad \text{Equation A-3.9}$$

where $\sigma_{\ln X} = \sqrt{Var[\ln X]}$, and $E[\ln X]$ is the expected value(mean) of the natural logarithim of X.

A-3.4.2.3 Probabilistic Measure of Slope Stability

In reliability-based analysis of slope stability, the input parameters that are not well defined are considered to vary according to some form of distribution as described in the previous section. These variable parameters are then used as input into a series of stability analyses to obtain the overall distribution of the performance function. The performance function is used to report the stability of the slope. The performance function used throughout this study for slope stability is the factor of safety.

A hypothetical distribution of the factor of safety that could result from analyses using probabilistic parameters is shown in Exhibit A-3.4 at the end of this chapter. As shown in the figure, the distribution indicates that the actual factor of safety may take on a range of possible values, ranging from well below the limiting value of FS = 1.0 to well above the limiting value. While knowledge of the complete distribution of the factor of safety is useful, it is the relative frequency of factors of safety less than the limiting value that are of primary importance (FS ≤ 1.0 => Failure). Three different probabilistic parameters are typically used to represent this relative frequency.

The probability of failure of a system is the area under the probability density function shown as the shaded area in Exhibit A-3.4. For the log-normal function, this would be from the boundaries (0 ≤ FS ≤ 1). In mathematical terms it can be expressed as:

$$P_f = \int_0^1 f_X(x)\,dx \qquad \text{Equation A-3.10}$$

where $f_X(x)$ is the probability density function expressed in Equation A-3.8.

The reliability of a system is conversely the area under the probability3 density function bounded by the limiting value and positive infinity. In Exhibit A-3.4, it is represented by the non-shaded area under the curve. For a log-normal distribution, the boundaries would be (1< FS ≤ +∞). Since the total probability for all possible values of the random variable is 1.0, the probability of failure, P_f, and the reliability, denoted as R, are related by:

$$P_f = 1 - R \qquad \text{Equation A-3.11}$$

Based on the assumption that the factor of safety is log-normally distributed, the natural log of the factor of safety will be normally distributed. In this case, the boundaries for the probability of failure would be ($-\infty < \ln FS \leq 0$). Under this assumption, the probability curve and its probabilistic parameters would be represented in Exhibit A-3.5 with the probability of failure in the shaded area. Exhibit A-3.5 can be found in the Supplemental Exhibits section at the end of this chapter.

The reliability index, β, is a gage of the reliability of a system that takes into account technicalities of the procedure and the uncertainties introduced by random input variables. The reliability index gives a measure of comparative reliability for a system, thereby making it unnecessary to calculate or determine the actual probability distribution. It is defined using the probabilistic terms of standard deviation and the expected value (mean) of the performance function. Graphically, the reliability index multiplied by the standard deviation is equal to the distance from the expected value (mean) to the limiting state as shown in Exhibit A-3.4. For a log-normal distribution, the reliability index is computed as:

$$\beta = \frac{\ln\left[E[FS] \Big/ \sqrt{1 + COV[FS]^2} \right]}{\sqrt{\ln\left(1 + COV[FS]^2\right)}} \qquad \text{Equation A-3.12}$$

where β is the reliability index, *E[FS]* is the expected value (mean) of the factor of safety, and *COV[FS]* is the coefficient of variation of the factor of safety.

A-3.4.2.4 Probabilistic Measure of Stability for Underseepage

When the excess head at the ground surface on the landward side of the levee toe is greater than zero and the blanket material is thicker than one-fourth the levee height, the probability of failure can be calculated using the method described in ETL 1110-2-556.

Using this method, the exit gradient (i) is assumed to be a log-normally distributed random variable with probabilistic moments $E[i]$ and σ_i. Based on this assumption, the equivalent normally distributed random variable has moments $E[\ln i]$ and $\sigma_{\ln i}$. The limit state for the underseepage would then be the natural log of the failure gradient (i_f) with the boundaries for the probability of failure being:

$$P_f = P(\ln i > \ln i_f) \qquad \text{Equation A-3.13}$$

The probability of the ln i being greater than the ln i_f is determined by using the standard normalized variate (z), which is also analogous to the reliability index β. The standard normalized variate is calculated as:

$$z = \beta = \frac{\ln i_f - E[\ln i]}{\sigma_{\ln i}} = \frac{\ln\left[\frac{i_f * \sqrt{1+COV[i]^2}}{E[i]}\right]}{\sqrt{\ln(1+COV[i]^2)}} \qquad \text{Equation A-3.14}$$

where, *E[i]* is the expected value (mean) of the hydraulic gradient and *COV [i]* is the coefficient of variation of the hydraulic gradient. Exhibit A-3.6 at the end of this chapter shows a graphical representation of the probabilistic parameters for the underseepage analysis with the probability of failure in the shaded area.

A-3.4.2.5 Taylor Series Approximation Method for Determining Risk and Uncertainty Analysis

As described in the previous sections, the probability of failure can be computed if the expected value (mean) and variance of the distribution are known. Numerous methods are available for computing the probability of failure for reliability-based analyses, including first order second moment methods (FOSM), the point estimate method, the Hasofer-Lind method, and Monte Carlo simulations (Baecher & Christian 2000). While all of these methods can be used, the most commonly used method to date in geotechnical applications is the Taylor Series Approximation of the FOSM method (USACE, 1999). The basis of the Taylor series method is that it uses the first two linear terms on the Taylor series expansion of the performance function to determine the probabilistic measures of performance. As such, the method is exact for linear performance functions and is approximated for higher order functions. While this method is approximate from a strictly probabilistic point of view, it has the significant advantage of being relatively simple to implement.

For a function (Y) of random independent variables ($X_1, X_2, \ldots X_n$) of the form

$$Y = g(X_1, X_2, \ldots X_n) \qquad \text{Equation A-3.15}$$

the expected value (mean) of Y can be found by evaluating the function at the expected values (mean) of the random variables. In the slope stability analysis application, the function Y is chosen to be the factor of safety and the random variables are the input parameters that are chosen as probabilistic. The expected value of the factor of safety is therefore computed directly from the expected values (mean) of the random variables.

Stated in mathematical form, this is:

$$E[FS] = FS(E[\bar{\phi}_{\text{foundation}}], E[\bar{\phi}_{\text{blanket}}], E[\bar{\phi}_{\text{embankment}}]) \qquad \text{Equation A-3.16}$$

where E[FS] is the expected value (mean) of the factor of safety and E[$\bar{\phi}$ $_{foundation}$], E[$\bar{\phi}$ $_{blanket}$], and E[$\bar{\phi}$ $_{embankment}$] are the expected values (mean) of the random variables.

The Taylor Series approximation for the variance of the factor of safety can be expressed as:

$$\text{Var[FS]} = \sum\left[\left(\frac{\partial FS}{\partial X_i}\right)^2 Var[X_i]\right] \qquad \text{Equation A-3.17}$$

where X_i represents a value of the ith random variable for the stability analysis, $Var[X_i]$ is the variance of that random variable, and $\frac{\partial FS}{\partial X_i}$ is the partial derivative of the distribution of the factor of safety evaluated at the expansion point. Noting that the $Var[X] = \sigma_X^2$ and approximating the partial derivative with a difference form, Equation A-3.17 becomes:

$$\text{Var[FS]} = = \sum\left(\left[\frac{\Delta FS}{\Delta X_i}\right]^2 \sigma_i^{\ 2}\right) \qquad \text{Equation A-3.18}$$

where σ_i is the standard deviation of the ith random variable and $\frac{\Delta FS}{\Delta X_i}$ is the approximated partial derivative. It has become common to evaluate the partial derivative $\frac{\Delta FS}{\Delta X_i}$ at the expected value (mean) plus one standard deviation and at the expected value (mean) minus one standard deviation as shown in Exhibit A-3.7, at the end of this chapter, so that $\Delta X_i = 2\sigma_i$. Making this simplification, the expression for the variance becomes:

$$\text{Var[FS]} = \sum\left(\frac{FS(E[FS]+\sigma_{FS}) - FS(E[FS]-\sigma_{FS})}{2}\right)^2 \qquad \text{Equation A-3.19}$$

where $FS(E[FS] + \sigma_{FS})$ is the factor of safety calculated at the expected value plus one standard deviation and $FS(E[FS] - \sigma_{FS})$ is the factor of safety calculated at the expected value minus one standard deviation. Noting that the $\sqrt{Var} = \sigma$, the equation for the standard deviation for the factor of safety will become:

$$\sigma_{FS} = \sqrt{\left(\frac{\Delta FS_1}{2}\right)^2 + \left(\frac{\Delta FS_2}{2}\right)^2 + \ldots\ldots + \left(\frac{\Delta FS_n}{2}\right)^2} \qquad \text{Equation A-3.20}$$

where σ_{FS} is the standard deviation of the factor of safety and ΔFS is the difference between the factors of safety calculated at the expected value plus and minus one standard deviation for each of the random variables.

The discussion above describes how the factor of safety was evaluated as the limit state function. The exact same procedure can also be used with the critical hydraulic gradient as the limit state with different input parameters applicable to the underseepage analysis.

Once the standard deviation and expected value for the factor of safety are known, the coefficient of variation (COV) for the factor of safety may be calculated and then used in Equation A-3.12 to compute the reliability index. Given the reliability index, β, the probability of failure is calculated using the built-in function NORMSDIST in Microsoft Excel. This function uses the reliability index as the argument allowing for the probability of failure to be computed as:

$$P_f = 1 - \text{NORMSDIST}\ (\beta) \qquad \text{Equation A-3.21}$$

A-3.4.3 Uncertainty Analyses

A-3.4.3.1 General

Risk-based analyses for the Armourdale Levee Unit were performed for the existing conditions. In these reliability analyses, geotechnical uncertainties were assessed by determining probability distributions for the blanket thickness and soil material properties for typical levee sections representative of the Armourdale Levee Unit.

Two types of geotechnical failures were analyzed:

1. A slope failure, defined as failure of the landside embankment slope resulting in water from the river flowing to the landside areas of the levee resulting in economic damages to the interior.
2. An underseepage failure, defined by excessive seepage initiating a levee failure and resulting in economic damages to the interior. Geotechnical failures may occur when river stages reach elevations at or below the top of levee.

The probability of failure of the levee is also conditional on the uncertainties associated with the hydrologic and hydraulic aspects of determining the water surface profile during a flood. These uncertainties can be combined with the geotechnical uncertainties and used in the HEC-FDA program. This is performed for economic purposes through the development of a relationship between the probability of failure of the levee and the height of water on the levees.

A-3.4.3.2 Probabilistic Underseepage Analysis

The actual conditions indicative of an underseepage failure are highly speculative. The underseepage analysis included in ETL 1110-2-556 - Appendix B uses a threshold value of gradient factor of safety of 1.0 to define failure. A gradient factor of safety of 1.0 reflects a condition where floatation of particles theoretically begins and seepage and boils can first physically occur, however it is not necessarily a condition indicative of having certain levee failure. Observations during the Flood of 1952 on the Missouri River are shown in Table A-3.3. The table shows the relation between observed field performance and calculated factors of safety. From the observations it can be seen that somewhere between a factor of safety of 0.55 and 0.80, undesirable seepage reaches a point where a failure could occur without outside intervention in the form of flood fighting. In an effort to define a condition more representative of actual levee failure due to underseepage for this study, a gradient safety factor of 0.70 was utilized as a threshold value for when certain levee failure is likely to occur. The chosen threshold value of gradient factor of safety of 0.70 falls within the "transition" zone in Table A-3.3 between tolerable seepage and objectionable seepage. In the probabilistic underseepage analyses a failure gradient (i_f) was calculated as:

$$i_f = \frac{i_c}{FS} = \frac{0.86}{0.70} = 1.23 \qquad \text{Equation A-3.22}$$

where i_c is the critical gradient and FS is the gradient safety factor. The factor of safety that defines failure was used to define the failure gradient in Equation A-3.22 and the limit state in Equation A-3.13.

TABLE A-3.3
Observations of Seepage Conditions During 1952 Flooding on the Missouri River at the Kansas Citys Flood Control Project

Computed Safety Factor at Flood Crest	Seepage conditions during flood Crest
Less than 0.55	Objectionable seepage: major flood fight; boils requiring sandbagging
0.55 to 0.80	Transition zone
Greater than 0.80	Tolerable seepage: distributed seepage, pin boils

The Kansas City District method of estimating the hydraulic gradients due to underseepage is slightly different than the method described in the EM 1110-2-1913. It is based on the findings made at the Missouri River Division Conference held by the Corps of Engineers in 1962 in Omaha. The underseepage analysis was based on experience during the flood event in 1952 along the Missouri River. The main differences in the Kansas City District method are:

1. The Kansas City District Method uses permeability ratios (See Table A-3.4.) related to differing material types of the blanket material instead of using actual horizontal and vertical permeabilities.

2. The Kansas City District Method assumes an infinite landside blanket in the analysis.

3. The Kansas City District Method does not use a transformed thickness for the soil stratum considered as EM 1110-2-1913 allows, instead, a representative permeability ratio is applied to the overall blanket thickness.

TABLE A-3.4
Permeability Ratios for Blanket Material Based on Material Type

Blanket Material	Assumed Permeability Ratio
SM	100
ML	200-400
ML - CL	400
CL	400-600
CH	800-1000

Additional information concerning the underseepage analysis for the Kansas City procedure can be found on the District's website at http://www.nwk.usace.army.mil/Portals/29/docs/construction/underseepage1.pdf.

The critical section for an underseepage failure along the Armourdale Levee Unit was chosen by calculating the expected value of the factor of safety with respect to hydraulic gradient at the toe of the levee for the entire unit. The reach with the lowest expected factor of safety was chosen for a risk analysis.

In the probabilistic analyses of underseepage using the Kansas City District method, three random variables were considered: blanket thickness, the permeability ratio and thickness of the aquifer.

Using existing subsurface information, it was assumed that the COV of the blanket thickness and thickness of the aquifer was 20 percent and 15 percent, respectively. These values for COV are deemed appropriate for the level of information available.

Using the published value given in ETL 1110-2-556, it was assumed that the COV of the permeability ratio was 40 percent. The permeability ratios used in the analyses followed the Kansas City District Guidance based on the type of material making up the blanket layer. In the existing conditions phase of the study the permeability ratios used in the underseepage analyses were based on material descriptions obtained from historical borings information from the Armourdale unit. Table A-3.4 lists the permeability ratios.

The underseepage analyses are then performed using the expected values of the random variables and plus and minus one standard deviations at different river levels. Using the log normal distributions and the limit state function for underseepage, a probability of failure can be developed for each river level at the critical locations.

A-3.4.3.3 Probabilistic Slope Stability Analysis

The conditions leading to a stability failure are less uncertain than those of an underseepage failure. A threshold value of stability factor of safety of 1.0 to define a slope failure is nearly universally accepted. The assumptions made for the slope stability component of the risk-based analysis allowed the evaluation to be more specific as to the magnitude of the failure and the actual consequences associated with that type of failure. The slope stability analyses assumed that the failure surface should be of significant magnitude to remove a major portion of the levee allowing the interior of the levee unit to flood.

The critical section for the stability analysis was chosen based on levee height and side slope steepness. The section with the tallest levee height and steepest side slopes was chosen for the probabilistic analysis.

Each zone of material making up the critical cross section of the levee was considered homogenous. The zones were comprised of three areas: the foundation sands, the blanket materials, and the embankment material. The foundation sand strengths were considered constant in the analysis. The piezometric surface through the levee cross section was simplified and considered to be in a steady state condition. The model that was used assumed that the water surface entered the slope at the point on the riverside where the river intersected the upstream slope face. The piezometric surface then continued in a linear path to the landside levee toe.

The soil strength parameters considered in the existing conditions analysis were modeled with drained strengths because steady seepages conditions were considered. The mean values and coefficients of variations were computed from raw data located in Design Memorandum No 3. The raw data used in this study was taken from consolidated drained direct shear tests performed for the 1962 Modification of the Armourdale Unit. The effective stress failure envelopes for normal effective stresses less than 2000 psf were used to characterize the strengths of the soils. This was done because the "working load" effective stresses in the embankment and foundation materials are generally near, or less than, this value during flood conditions.

The materials evaluated were designated as either foundation blanket material or embankment fill material. Based upon available laboratory test data, with the results shown in Table A-3.5, it was determined that the blanket had an expected value ($E[\bar{\phi}]$) of 33° with a coefficient of variation ($COV_{\bar{\phi}}$) of 16 percent, and the embankment had an expected value ($E[\bar{\phi}]$) of 32° with a coefficient of variation ($COV_{\bar{\phi}}$) of 14 percent. Cohesion (c) was assumed to be zero with no variation for both materials.

The pore pressures developed in the blanket material were determined from the hydraulic gradient calculated at the base of the blanket material due to underseepage. The hydraulic gradient line was based on the output from the underseepage analysis using the Kansas City District Method. Assuming that the elevation head datum is at the same

TABLE A-3.5
Effective Strength Data Used for Risk and Reliability Analysis
Embankment and Foundation Materials

Boring	Sample	Soil	Material	τ (tsf)	σ (tsf)	ϕ (degrees)
U-549	3	Sandy Clay	Foundation	0.67	1.0	33.8
U-549	Wax-9a	Silt	Foundation	0.73	1.0	36.1
U-549	Wax-10	Silt	Foundation	0.63	1.0	32.2
U-549	Wax-8a	Silt	Foundation	0.41	1.0	22.3
U-549	Wax-8b	Silt	Foundation	0.67	1.0	33.8
U-549	Wax-9b	Silt	Foundation	0.56	1.0	29.2
U-549	Wax-10	Silt	Foundation	0.71	1.0	35.4
U-550	Wax-1a	Lean Clay	Foundation	0.67	1.0	33.8
U-550	Wax-1b	Lean Clay	Foundation	0.77	1.0	37.6
U-550	Wax-2	Lean Clay	Foundation	0.60	1.0	31.0
U-550	Wax-8a	Lean Clay	Foundation	0.67	1.0	33.8
U-550	Wax-8b	Sandy Silt	Foundation	0.79	1.0	38.3
U-550	Wax-4	Lean Clay	Foundation	0.71	1.0	35.4
U-550	Wax-9	Lean Clay	Foundation	0.66	1.0	33.4
U-550A	Wax-2b	Sandy Clay	Foundation	0.71	1.0	35.4
U-550A	Wax-2a	Lean Clay	Foundation	0.67	1.0	33.8
U-551	Wax-2	Fat Sandy Clay	Foundation	0.63	1.0	32.2
U-551	Wax-3	Fat Clay	Foundation	0.59	1.0	30.5
U-551	Wax-4	Sandy Silt	Foundation	0.66	1.0	33.4
U-551	Wax-5	Fat Sandy Clay	Foundation	0.55	1.0	28.8
U-551	Wax-6	Fat Sandy Clay	Foundation	0.41	1.0	22.3
U-551	Wax-6	Fat Clay	Foundation	0.41	1.0	22.3
U-552	Wax-4a	Fat Sandy Clay	Foundation	1.00	1.0	45.0
U-552	Wax-4b	Fat Sandy Clay	Foundation	0.99	1.0	44.7
U-552	Wax-5b	Fat Organic Clay	Foundation	0.71	1.0	35.4
U-552	Wax-5a	Fat Clay	Foundation	0.59	1.0	30.5
U-552	Wax-6	Fat Clay	Foundation	0.59	1.0	30.5
D-530	sk-2	Lean Clay	Embankment	0.49	1.0	26.1
A-534	sk-2	Lean Clay	Embankment	0.45	1.0	24.2
A-537	sk-2	Lean Clay	Embankment	0.67	1.0	33.8
A-537	sk-3	Fat Clay	Embankment	0.59	1.0	30.5
D-539	2	Lean Clay	Embankment	0.71	1.0	35.4
D-539	3	Silt	Embankment	0.71	1.0	35.4
HA-539	5	Fat Clay	Embankment	0.62	1.0	31.8
543	7	Lean Clay	Embankment	0.73	1.0	36.1

elevation as the base of the blanket material, the pore pressure (u) at a point along the base of the blanket material would be equal to the distance from the hydraulic gradient line (h_p) to the base of the blanket multiplied by the unit weight of water (γ_w). The mathematical relation can be stated as follows:

$$u = h_p * \gamma_w \qquad \text{Equation A-3.23}$$

For points within the slope, the pore pressure at the top of the blanket was calculated as the distance from the phreatic surface to the top of the blanket (h_p) multiplied by the unit weight of water (γ_w) (as in Equation A-3.23). The pore pressure at the base of the blanket was calculated using the distance from the hydraulic gradient line as the pressure head (h_p) in Equation A-3.23. A linear interpolation between these two pore pressures would give the pressure distribution through the blanket material used in the slope stability analysis.

The embankment was assumed to be homogenous and impervious, even though it is comprised of impervious and random zones. This was done to simplify the analysis and due to the fact the random material is mostly comprised of impervious material.

The slope stability analyses were carried out in the same manner prescribed in ETL 1110-2-556. Utilizing the slope stability program UTEXAS 4 (using Spencer's Method), an initial circular search was performed using the expected values (means) for the random variables considered in the analysis. In order to determine a surface that would mobilize a large portion of the embankment that would lead to a catastrophic failure, a series of single surface searches were performed to locate the critical surface. The failure surface was forced through the intersection of the water surface and the slope face to model a catastrophic failure that would cause interior flooding. Using this boundary condition, the failure would be of significant magnitude to inundate the levee interior instead of assuming a progressive slope failure from the landward levee toe.

An initial run in the UTEXAS 4 program was made using the expected values $E[\bar{\phi}]$ for each of the different material types. The factor of safety (FS) obtained from this analysis gave the expected value for the factor of safety E [FS]. The failure surface obtained from this initial run was then considered the critical surface. The remaining series of runs were made at plus and minus one standard deviation of the expected values for strength along the critical surface defined in the initial run. As each material property was changed, a resulting factor of safety was computed. The variation resulting in each change for that particular material type can then be used in the Taylor Series Approximation. Using the probabilistic methods described previously, a probability of failure could be determined for a specific river elevation. The procedure was then repeated for various river levels and a probability curve was computed based on slope stability relationships with river levels.

A-3.4.4 Results for the Reliability-Based Analyses of the Kansas Citys – Missouri and Kansas Flood Risk Management Project

A-3.4.4.1 Underseepage Results

The critical section for the Armourdale Levee Unit with respect to an underseepage failure was computed to be the "slot" area at approximately Station 276+00 under the railroad bridges. This section was chosen as the critical section because it had the lowest expected value for factor of safety (0.90) for the entire unit. The "slot" area is comprised of an old railroad bed, up to 15-ft lower than the surrounding ground, directly adjacent to the landside of the existing floodwall. There is a system of fully penetrating artesian relief wells in the "slot" area that discharge directly into the slot. The purpose of the wells is to reduce the pressure at the base of the blanket and to fill the slot with water to further reduce the gradient through the blanket. The original design condition for the slot area is for the slot to be filled with a minimum of 5-ft of water during the maximum flood water level. The slot is to be maintained inundated with a minimum of 5-ft of water until the river levels recede. It is highly unlikely that a flood of any magnitude would occur and a minimum of 5-ft of water would not be ponded in the slot area. The operational restriction and required ponding levels are detailed in the Operation and Maintenance Manual and Design Memorandum No. 3.

The typical section used in the analysis consisted of 18.5 ft of driving head, and an expected value for blanket thickness in the slot of 14-ft. The expected value for permeability ratio and foundation sand depth is 300 and 47-ft, respectively.

The calculation necessary to determine the probability of failure for the slot area, which would include the uncertainties in the well flows, would be computationally intense. So the probability of a levee failure due to piping in the "slot" area was calculated with some deviations from the method described in the probabilistic underseepage analysis discussion due to the relief wells in the "slot" area. First, the expected value of the factor of safety was calculated, including the effects of the relief wells and interior ponding, for varying river stages for the slot area. The information used to calculate the expected value of the factor of safety is provided in Table A-3.6. To approximate the probability of failure from the calculated expected value of the factor of safety considering the relief wells, a relation between expected value of the factor of safety and probability of failure was developed using the methods described in the probabilistic underseepage analysis discussion that does not consider the relief wells. The statistical parameters described above for the coefficient of variances and threshold values were used in the determination of the relation. The relation, shown in Exhibit A-3.8 at the end of this chapter, was then used to determine the probability of failure using the calculated expected value of the factor of safety.

The probability of a levee failure due to piping in the "slot" area for the existing condition is shown in Exhibit A-3.9 at the end of this chapter. At the maximum river level, during steady state seepage conditions, the probability of failure is 8%. It should be noted that the probability of failure due to a piping failure in the "slot" area greatly increases if the slot is not allowed to fill with a minimum of 5-ft of water as it was designed to operate. Naturally, allowing the slot to fill with more than 5-ft of water decreases the probability of failure, as additional water on top of the blanket decreases the gradient through the blanket. Field visits indicate that the slot would be able to hold approximately 10-ft of water with no adverse affect to the protected area.

TABLE A-3.6
Expected Value of Factor of Safety for Piping – Station 276+00

River Elevation (ft)	HGL @ Base of Blanket (ft)	h_0 w/o Ponding (ft)	Flow into the Slot (cfs)	Likely Depth of Ponded Water (ft)	h_0 w/ Ponded Water	i	$FS_{iexpected}$
753	751.6	12.6	2.6	5.0	7.6	0.54	1.55
755	752.7	13.7	3.3	5.0	8.7	0.62	1.35
757	753.8	14.8	4.0	5.0	9.8	0.70	1.20
759	754.9	15.9	4.7	5.0	10.9	0.78	1.08
761	756.3	17.3	5.4	5.0	12.3	0.88	0.96
762.5	757.0	18.0	6.0	5.0	13.0	0.93	0.90

i_c calculated with a blanket unit weight of 115 pcf
Top of Blanket in Slot = 739 ft
Bottom of Blanket in Slot = 725 ft
Volume of Slot = 195,000 ft^3
Probability of Failure taken from Probability of Failure vs. Expected Factor of Safety

A-3.4.4.2 Stability Results

The critical section for the Armourdale Levee Unit with respect to slope stability was located at approximately Station 222+00. This section was chosen as the critical section due to the levee height and levee side slopes.

The levee at Station 222+00 has a typical cross section of a 17.5-ft high levee with a side slope of 2.5:1 (horizontal to vertical) on the riverside, a crest width of 10-ft, and a net side slope of 2.5:1 (horizontal to vertical) on the landward side. The net landside side slope is comprised of a series of slopes and retaining walls that result in a net slope of approximately 2.5:1.

The probability of failure due to slope stability is shown in Exhibit A-3.10 at the end of this chapter. At the maximum river level, during steady state seepage conditions, the probability of failure was calculated to be 24%.

A-3.4.5 Summary

The geotechnical existing conditions analysis was performed to identify the critical sections from a geotechnical perspective and determine their probability of failure. The probabilistic analyses performed for this study were modeled with guidance given in ETL 1110-2-556 "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies" (28 May 1999).

Two modes of unsatisfactory performance were considered at various river stages-underseepage and landside slope stability under a steady state seepage condition. Where enough information was present, the probabilistic parameters needed for each of the variables as calculated. If little or no raw data was available, assumptions were made based on work done by others in the field of geotechnical risk-based analysis.

A-3.5 DETERMINISTIC AND ANALYSIS METHODS FOR DESIGN OF NEW FEATURES

A-3.5.1 Slope Stability Criteria

For the Kansas City Levees Phase 2 Feasibility Study, three proposed methods for raising the line of protection were analyzed for stability. These included an earth fill landside raise, an earth fill riverside raise, and a cantilever retaining wall raise on top of an existing levee. The criteria used for the slope stability analysis was from Engineering Manual 1110-2-1913, Design and Construction of Levees, dated April 2000. The engineering manual lists the following minimum requirements in Table A-3.7 with respect to a deterministic slope stability analysis:

Table A-3.7
Minimum Factors of Safety

Loading Condition	Minimum Factor of Safety
End of Construction	1.3
Steady Seepage	1.4
Rapid Drawdown	1.0 to1.2*

*Lower factors of safety may be appropriate when the consequences of failure in terms of safety, environmental damage, and economic losses are small

The end of construction and steady seepage cases were analyzed for this Feasibility Study. The rapid drawdown stability analysis was not performed due to the lack of required shear strength parameters for the two stage analysis. Additional drilling and testing will be required as part of PED to determine the shear strength parameters for this analysis.

For levees in an urban area, rapid drawdown failure could be significant in terms of economics, not only for the temporary loss of protection but also for repairs to the levee. It is recommended that a factor of safety of 1.2 be used for this failure condition. The engineering manual does not specify the water levels for the loading condition, so the assumptions in Table A-3.8 were used.

Table A-3.8 – Water Loading Conditions

Loading Condition	Water Level for Stability Analysis
End of Construction	Water at Top of Natural Blanket
Steady Seepage	Water at Top of Protection, Riverside
Rapid Drawdown	Water at Top of Protection, Stage 1 Water at Landside Toe of Levee, Stage 2*

*Or landside ground elevation, whichever is lower

Guidance was published by the HQUSACE in April 2007 with respect to Hurricane Protection System slope stability design criteria guidance. The document was published in the form of a Memorandum to the Commander, Mississippi Valley Division, and intended for use during levee rehabilitation in Southeast Louisiana. The revised design criteria was based on criteria presented in EM 1110-2-1902 Slope Stability, dated Oct 2003, for new embankment dams. The original criteria are consistent with that presented in Table A-3.7. The new guidance suggests a factor of safety of 1.5 (if the site conditions are "well defined") for what is called the "extreme hurricane" condition, when steady state conditions are expected to develop with water at the top of protection. This is an increase in factor of safety from what was used for this feasibility study. Though the published criteria are currently only related to hurricane loadings, it could easily be transferred to all levees in the future. It is suggested that slope stability criteria be reviewed and revised as necessary during PED. If increased factors of safety are required for the Armourdale unit, the implications would likely consist of additional required real estate for expansion of stability berms.

A-3.5.2 Underseepage Criteria and Analysis

The current Corps of Engineers guidance on underseepage is contained in Engineering Technical Letter (ETL) 1110-2-569. The ETL recommends using all definitions, design equations, and procedures in Engineering Manual (EM) 1110-2-1913 except as noted within. The greatest deviation from the EM is the requirement for a maximum hydraulic gradient through the landside blanket at all points landward of the levee of 0.5, which provides for a factor of safety with respect to hydraulic gradient (FS_i) of approximately 1.6. For the design of future conditions alternatives of the Armourdale Unit, the criterion shown below was used to determine whether underseepage control measures are necessary.

With water at the top of line of protection:

- FS_i equal to, or greater than, 1.6 – No underseepage control measures are necessary.
- FS_i less than 1.6 – Design underseepage control measures to achieve a FS_i = 1.6.

The general procedure outlined in EM 1110-2-1913 Design and Construction of Levees was used to calculate the factor of safety with respect to hydraulic gradient for the natural blanket, and to calculate the excess head at the landside toe (assumed to be acting at the bottom of the blanket) of the line of protection. The variations from EM 1110-2-1913 used in the analysis is as follows and discussed previously in this chapter:

1. The use of permeability ratios relating to different material types for the natural blanket, as opposed to actual horizontal and vertical permeabilities.

2. The assumption of an infinite landside blanket.

3. No blanket thickness transformation is performed.

The general procedure outlined in EM 1110-2-1914 Design, Construction, and Maintenance of Relief Wells, Figure 5-3 was used to analyze and design all relief well systems. The variations from EM 1110-2-1914 used in the analysis and design are:

1. The excess head computed at the landside toe was used as the net head on the system of wells instead of full driving head. This was done because the procedure outlined in Figure 5-3 assumes an impervious blanket. However, a semi-pervious blanket was assumed for the underseepage calculations.

2. An efficiency reduction factor of 0.8 was applied to the expected well flows. This was done to account for the reduction in efficiency with time of the relief wells. An efficiency factor of 0.8 was chosen as EM 1110-2-1914 requires remedial action once a loss of 20% in specific capacity of a well is observed from pumping test.

A-3.6 N500+3 STABILITY ANALYSES

A-3.6.1 Sections Analyzed

For the Armourdale Unit, the design team developed four lines of protection raise configurations to raise the level of protection to an N500+3 event on the Kansas River. The method for computing the river stage for this event is discussed in the Hydrology and Hydraulics chapter (Chapter A-2) of the Phase 1 Feasibility Report. The top of the proposed raise was set approximately equal to the river stage for the N500+3 event. The four raise configurations are as follows:

1. Landside earth fill raise
2. Riverside earth fill raise
3. Cantilever floodwall on top of the existing levee
4. Floodwall

All levee sections will maintain a 10-ft crest width to maintain the current level of vehicle access. Floodwall stability is addressed in the structural chapter of this Feasibility Report.

To begin the evaluation process, a site visit was made to Armourdale to make an initial attempt to identify the most appropriate raise configuration along the entire unit on a reach by reach basis. Based upon the initial assignments the next step was to identify the most critical cross sections to analyze. The critical sections were selected by considering the existing levee height, height of proposed raise, thickness of the natural blanket, and the results of the underseepage analyses. Two cross sections were analyzed for each of the proposed raise configurations. Calculations are provided in Exhibit A-3.11 at the end of this chapter. A summary of existing levee stationing, existing heights, proposed raises, and recommended section is provided in Exhibit A-3.12.

The computer program UTEXAS4, developed by Stephen G. Wright of the University of Texas at Austin, was used to perform the analyses. The selected analysis method was Spencer's method, which is a limit-equilibrium approach that satisfies both force and moment equilibrium. The program has the ability to "search" for the critical failure surface with the lowest factor of safety for the given input parameters. As stated previously, only the end of construction and steady seepage loading conditions were analyzed. Steady seepage conditions controlled the final section dimensions for all sections analyzed. Potential rapid drawdown failure of the riverside slope will also have to be evaluated after additional geotechnical laboratory testing can be performed to determine the necessary strength parameters required for this analysis.

A-3.6.2 Landside Earth Fill Raise

A landside earth fill raise is the preferred raise configuration due to the low cost and ease of construction, and was proposed wherever possible. There are several reaches of the unit where this raise configuration was proposed. Two sections were selected to be analyzed. One section was selected due to the high piezometric levels in the foundation, and the other section was selected because it was the tallest levee section. The levee section was raised by maintaining the riverside slope and shifting the levee centerline landward.

The first landside fill section analyzed was at Station 90+00. This section has the highest piezometric levels in the foundation of the reaches where the landside fill was proposed. The proposed height of the levee for the N500+3 raise is 16.5-ft. The stability analysis indicated an acceptable cross section requires a 1V on 4H landside slope. The existing levee cross section has a 1V on 3H landside slope. A typical cross section for this location is shown in Exhibit A-3.13 at the end of this chapter, and is labeled Section 1. A drainage layer was added under the new landside fill to improve the internal seepage conditions in the embankment.

The second landside fill section analyzed was at Station 245+50. This section was the tallest proposed section for the landside raise configuration. The proposed height of the levee is 20-ft. The general approach for analyzing this section was the same as for Section 1. For this section to meet the minimum factor of safety two stability berms were required to be added to the basic levee section developed for Station 90+00; a 20-ft wide, 8-ft tall berm with a 1V on 4H slope and a shorter 10-ft wide, 4-ft tall berm with a 1V on 3.5H slope. A typical cross section for this location is shown in Exhibit A-3.13 at the end of this chapter, and is labeled Section 2. Again, a drainage layer was added under the new landside fill to control the internal seepage conditions in the embankment.

There are several reaches of the levee with a proposed landside raise configuration that have new levee heights between 16.5-ft and 20-ft. For the feasibility levee design it was assumed that the section for Station 245+50 would apply to these reaches. It is suggested that for future work that at least one intermediate section be analyzed to optimize the cross sections in these reaches.

A-3.6.3 Riverside Earth Fill Raise

A riverside earth fill raise is proposed at one location; at the upper end of the project between Station 3+25 UE and 10+00 UE. At this location there are real estate constraints on the landside of the current levee and a wide foreshore area on the riverside. The riverside fill levee section configuration is raised by maintaining the landside slope, so the levee centerline shifts riverward.

The riverside fill section evaluated was at Station 7+00 UE. The analysis indicates a small stability berm is required on the landside due to high piezometric levels in the foundation from the steady seepage loading condition. A typical cross section for this location is shown in Exhibit A-3.13 at the end of this chapter, and is labeled Section 3. A drainage layer was added in the landside berm to control the steady seepage water surface through the embankment.

A-3.6.4 Cantilever Floodwall on Top of Existing Levee

Using a cantilever wall to raise the level of the existing levee section is proposed where minimal real estate impacts on the landward side of the current protection are preferred. The floodwall is placed in the levee section so the riverside face of the wall is at the riverward edge of the crest. The local wall stability (i.e. sliding, bearing capacity and overturning) is discussed in the structures chapter in this Feasibility Study Report. This discussion focuses on the overall global stability of the section. Critical failure surfaces were searched both riverward and landward of the wall. Again, two sections were analyzed for this raise configuration. The end of construction loading condition was not analyzed due to the limited amount of additional fill that would be placed in this section.

Utilizing a cantilever floodwall to raise a levee section presented a significant design challenge due to the piezometric water levels developed in the foundation and the embankment for the steady seepage condition. The piezometric water levels increase significantly over the existing condition levels, however due to the wall raise there is minimal additional levee section being added to balance the increase in seepage pressures. Because of this situation, additional landside fill was required to improve the stability of this section.

The first cantilever wall section analyzed was at Station 100+00. This section was selected because it is the largest height increase for this configuration, 5.2-ft. The existing height of levee (measured from the landside) is 10.5-ft. To meet stability criteria, the existing levee section was modified until the minimum acceptable factor of safety was obtained. The final proposed section consists of a 1-ft crest raise, a 10-ft wide crest width, and a 1V on 4H landside slope. A typical section for the cantilever wall section is shown in Exhibit A-3.13 and is labeled Section 5 thru 9. The section at Station 100+00 is section number 5. A drainage layer in the existing levee section was added at the elevation of the wall footing to control the steady seepage water surface through the embankment and minimize the amount of additional fill required.

The second cantilever wall raise section analyzed was at Station 122+50. This section was initially thought to have the tallest overall proposed section height, (This sentence is confusing as to its importance, maybe reword to clarify) a 14-ft existing levee height and a 4.8-ft raise. The section was analyzed using the section developed for Station 100+00 (section 5). However, a small stability berm is required for this taller section to meet stability criteria. A typical section for the cantilever wall section is shown in Exhibit A-3.13, and is labeled Section 5 thru 9. The section at Station 122+50 is shown in Exhibit A-3.13 and is labeled Section 7. A drainage layer in the existing levee section was added at the elevation of the wall footing to control the steady seepage water surface through the embankment and minimize the amount of additional fill required. Due to the additional landside fill required to meet the stability requirements of this section, the proposed raise configuration through this reach was changed to a floodwall. The results of this analysis are still valid, however, and were used for assigning sections to other similar reaches.

For the cantilever wall on existing levee configuration, the two sections evaluated were "interpolated" between to estimate the requirements of sections with existing levee heights intermediate between the two analyzed sections. The existing levee heights were broken into 2-ft intervals starting at < 10-ft. A table showing the section requirements for each interval is provided on Exhibit A-3.13. After the initial round of analyses a reach of the existing levee unit was found to be up to 18-ft in height. This is 4-ft taller than the tallest section previously analyzed. For feasibility study level design, the required retaining wall section for two additional intervals of existing levee heights were "projected" based upon the existing analysis. It is recommended that during the next phase of the project that these taller sections be analyzed to verify an acceptable section. The reach in question is between Station 228+00 and Station 245+00.

A-3.7 N500+3 UNDERSEEPAGE ANALYSES AND CONTROL FEATURES

A-3.7.1 Subsurface Information

The natural blanket was characterized using subsurface information obtained from Design Memorandum No 3. The memorandum contains the results of an extensive subsurface investigation that was performed for the design of the 1962 Modification of the Armourdale Unit. The subsurface investigation was used to identify the soils present in the foundation of the line of protection, and establish their geotechnical parameters. This information was used to determine blanket thickness and composition which was used in the underseepage analysis. The Armourdale Unit has a foundation blanket varying between 12-ft and 40-ft thick, consisting of silts, clays, and discontinuous sand lenses. Underlying the foundation blanket is between 50-ft and 70-ft of clean sand before bedrock is encountered.

A-3.7.2 N500+3 Underseepage Analysis

Raising the Armourdale Levee Unit to a N500+3 level of protection increases the water pressures in the foundation sands, which in turn increases the hydraulic gradient through the natural blanket material. For the underseepage analysis, the entire Armourdale Levee Unit was divided into reaches of similar protection height, blanket thickness, blanket composition, aquifer thickness, and seepage entrance conditions. The factor of safety with respect to hydraulic gradient through the blanket was calculated for each of these reaches at the toe of the line of protection, and other critical areas such as building foundations and low areas as necessary. If the calculated factor of safety with respect to hydraulic gradient was calculated to be greater than 1.6 at all locations landward of the line of protection, no remedial measures were proposed. If the calculated factor of safety with respect to hydraulic gradient was calculated to be less than 1.6, remedial measures were proposed that would achieve a factor of safety with respect to hydraulic gradient of 1.6 at all locations landward of the line of protection. The design condition is to have a factor of safety with respect to hydraulic gradient equal to, or greater than, 1.6 at all locations landward of the line of protection toe. Exhibit A-3.14 shows the calculated factor of safety with respect to hydraulic gradient for the entire Armourdale Levee Unit (without the effects of relief wells or cutoff walls), as well as the parameters used to calculate the factor of safety at the end of this chapter.

A-3.7.3 Special Features Analyzed for Underseepage

Several existing structures and special features were analyzed to ensure a factor of safety with respect to hydraulic gradient of 1.6 is available in the thinner blanket under the feature. Exhibit A-3.15, located in the supplemental exhibits section, provides details on all special features that were analyzed for the Armourdale Levee Unit. Building basement and feature elevations were assumed to be 10-ft below the ground surface if their elevations were not provided in DM3. In the calculation of the allowable excess head in the reduced blanket thickness under specified features, the differential head across the blanket was assumed to be the excess head above the surrounding ground surface. Essentially, the structures were analyzed as a void in the blanket filled with water to the elevation of the surrounding ground. This analysis method is valid as long as the basements are water tight, completely flooded, or as long as seepage into the basements is controlled or stopped. This approach ensures an acceptable gradient through the foundation blanket under the structure for the conditions noted above, however does not prevent potential seepage related problems into basements or structural problems due to large water pressures under basement slabs.

The increase in water pressure under basement slabs due to the N500+3 raise was calculated to determine the possible implications on the integrity of the structures. First, the existing conditions were analyzed to establish the baseline for any increase in pressure. The conditions analyzed for the 1962 Modification are the same as the current existing conditions, as the Armourdale Unit has not undergone significant modification since the 1962 Modification was analyzed. The existing conditions were modeled utilizing the same methods as the N500+3 analyses and compared to the calculations performed for the 1962 Modification at individual checkpoints. This comparison is shown in Exhibit A-3.16 at the end of this chapter. The initial hydraulic grade line (not considering well effects) calculated for the 1962 Modification and the Feasibility Study are nearly identical. This is because the same blanket theory equations used in 1962 are still in use today. However, the drawdown calculated for the 1962 Modification is an average of 2.2-ft greater than that calculated for the Feasibility Study at all checkpoints. A portion of the difference in calculated drawdown is attributed to the difference in well flow rates used in each calculation. The 1962 Modification calculations assumed a flow from each well of 1.33 cfs. The well flows for the Feasibility Study were calculated, and averaged approximately 1.18 cfs. This difference in well flows amounts to approximately a 0.5 difference in drawdown. The remaining difference of approximately 1.7-ft is likely attributed to differences in the drawdown calculation methods. Due to the difference of approximately 2.2-ft in the hydraulic grade lines at all checkpoints, the existing conditions at building locations was approximated by subtracting 2.2-ft from the that calculated for the Feasibility Study to capture the intent of the original designers in 1962.

The water pressure acting under basement slabs was calculated assuming a linear distribution of water pressure between the base of the blanket and the top of the blanket. The hydraulic grade line calculated in the underseepage analysis acting at the base of the blanket is used to calculate the water pressure acting at the base of the blanket. The water pressure is assumed to be zero at the ground surface.

The water pressure acting on the basement slabs are shown for the existing condition (1962 Modification and Feasibility Study Calculations), the N500+3 condition, and for a hydrostatic condition in Exhibit A-3.15 at the end of this chapter. The water pressure acting on the bottom of the feature increases by 1-ft of hydraulic head or less over the existing condition (1962 Modification Calculation) as a result of the N500+3 raise considering all underseepage control measures for both conditions assuming a linear pressure drop across the blanket. The effects of the increase in pressure caused by the N500+3 raise over the existing conditions are not considered to be significant. However, no analysis was performed to determine the structural integrity of the basement slabs under either existing conditions or the N500+3 raise. This should be further analyzed during final design, as the pressures developed during a flood event due to underseepage may be significantly greater than hydrostatic pressure.

A-3.7.4 Underseepage Control Requirements

The reaches outlined in detail in this section are either at the minimally acceptable factor of safety with respect to hydraulic gradient without remedial measures, or require remedial measures to increase the factor of safety to the minimally acceptable factor of safety with respect to hydraulic gradient. All existing underseepage control features on the Armourdale Unit outlined previously are assumed to remain in place as functional features, except as noted in the following discussion.

Station 66+00 to 79+00

This reach is characterized by an area with a locally thin natural blanket between 12 and 15-ft in thickness. The calculated factor of safety with respect to hydraulic gradient in this reach ranges from 0.8 to 0.9 with water at the N500+3 elevation. A slurry cutoff wall is proposed to remediate this underseepage concern. The slurry cut off wall should extend to bedrock (approximately 90-ft below the landside ground surface) and should hydraulically connect to an impervious section of the line of protection riverward of the centerline. The slurry cutoff wall needs to extend beyond the critical reach of Station 66+00 to 79+00 to negate the seepage that will occur around the ends of the cut off wall. To determine the extension of the wall beyond the critical reach necessary to achieve the minimally acceptable factor of safety, hydraulic grade lines calculated using the Kansas City District method for the reaches between Station 60+00 to 66+00 and Station 73+00 to 79+00 were used to determine the shortest allowable seepage path around the slurry cutoff wall that will result in the maximum allowable head at the critical points. The maximum allowable excess head to have the minimally acceptable factor of safety with respect to hydraulic gradient at Station 66+00 and 79+00 (the ends of the critical reach) is 6.5-ft and 7.8-ft, respectively. The hydraulic grade lines indicate that the slurry cutoff wall must extend beyond the critical reach approximately 400-ft beyond Station 66+00 and approximately 300-ft beyond Station 79+00. The slurry cutoff wall should be constructed between Stations 62+00 and 82+00. Exhibit A-3.17 shows the hydraulic grade lines that were used to determine the cutoff wall extensions beyond the critical area at the end of this chapter. It should be noted that there are some significant utilities that will have to be abandoned or modified to accommodate the slurry wall near Station 62+00, between Stations 75+00 and 77+00, and near Station 79+50. Additionally, the installation of a cutoff wall may change the overall groundwater flow in the area. The effects of the cutoff wall on the local ground water table were not considered. Should the effects on the local ground water table become an issue, surface discharging relief wells would become an acceptable alternative.

Station 86+00 to 93+00
This reach is comprised of an area with the minimally acceptable factor of safety with respect to hydraulic gradient of 1.6 at the landside toe with water at the N500+3 elevation. No remedial measures are proposed at this time.

Station 100+100 to 130+00
This reach is comprised of an area with the minimally acceptable factor of safety with respect to hydraulic gradient of 1.6 at the landside toe with water at the N500+3 elevation. No remedial measures are proposed at this time.

Station 157+00
There is a localized rectangular "ditch" landward of the Mill Street Pump Station situated perpendicular to the line of protection. The "ditch" is approximately 100-ft from the line of protection toe, and is approximately 13-ft lower than the adjacent ground. A property line transverses the ditch centerline, indicating that the "ditch" may be the remnants of separate fills placed on two adjacent parcels of property which were each sloped to the property line. In its current configuration, the calculated factor of safety with respect to hydraulic gradient is approximately 1.1 in the "ditch" with water at the N500+3 elevation. It is recommended that the ditch be filled with impervious material to the elevation of the surrounding impervious blanket (elevation 760) to ensure the minimally

acceptable factor of safety with respect to hydraulic gradient of 1.6 is achieved at all points landward of the line of protection.

Station 190+00 to 254+00

This long reach is comprised of a marginally thin blanket with zones of sand and fill in the upper portions of the blanket. This reach is also characterized by the many existing structures in close proximity to the line of protection. The existing relief well system between Stations 190+75 and 246+35 and aerial fill between Stations 220+00 and 226+50 was constructed during the 1962 Modification to alleviate minor underseepage concerns through the blanket and major underseepage concerns related to the existing building foundations. The existing relief well system is a series of 24 fully penetrating artesian relief wells connected by a header pipe which directs well flows to the Shawnee Pump Station. To provide a calculated factor of safety with respect to hydraulic gradient of 1.6 at all locations landward of the line of protection for the N500+3 water level, minor modifications to the existing system and additional fully penetrating artesian relief wells are proposed.

The proposed modifications to the existing system of 24 fully penetrating wells are very minor. The modifications will be limited to removing the extensions of the riser pipe above the lateral header pipe, and relocating the discharge elevation to the elevation of the header pipe at each relief well location.

A total of 39 new relief wells are proposed to be added to the existing relief well system between Stations 190+00 and 254+00 at the landside toe of the line of protection. All new wells were assumed to discharge at the ground surface to avoid additional pump station requirements. Exhibit A-3.18, located in the supplemental exhibits section, shows the locations of existing and proposed wells, along with the proposed discharge elevations and computed flow rates. Of the new wells, 25 wells with discharge elevations at the design landside ground elevation at each well location are required between Stations 190+00 and 246+00, mostly to protect the existing building foundations. The remaining 14 new wells are required between Stations 246+00 and 254+00 to ensure the minimally acceptable factor of safety with respect to hydraulic gradient of 1.6 through a regionally thin blanket. The wells between Stations 246+00 and 254+00 were designed using a conservative blanket thickness based on the available limited subsurface information on the riverside of the line of protection. Additional subsurface information should be obtained on the landside of the line of protection during final design to confirm that the wells between Stations 246+00 and 254+00 are required. Exhibits A-3.19, located in the supplemental exhibits section, shows the computed excess head at the landside toe of the line of protection between Stations 190+00 and 254+00 for the N500+3 water elevation with the existing and proposed relief wells. Exhibit A-3.20, located in the supplemental exhibits section, shows the computed excess head at the special features located in this same reach shown in Exhibit A-3.15.

The total flow from the existing 24 wells, after the proposed modifications, was computed to be approximately 47 cfs for the N500+3 water level. The Shawnee Pump Station, which services the wells, was originally designed for a well flow of 32 cfs. The pump station will have to be modified to continue servicing the wells. The capacity of the header pipe, however, is sufficient for the increased flows from the existing system. The total flow from the new 39 wells was computed to be approximately 40 cfs. The wells were designed to discharge at the landside ground elevation at the well location. All flow from the new wells will discharge at the ground surface and flow into existing interior drainage features.

All building basement elevations should be verified during future design. The relief well system should be refined at that time to incorporate the actual presence of basements and their elevations. The existing relief wells should be pump tested during final design to determine if they still perform adequately. If they do not, they should be replaced. If the existing wells must be replaced, it would be prudent to rework the well layout to economize the design.

During refinements of the well system during future design, it may be possible to economize and/or add reliability to the relief well system. Some required redundancies in the Shawnee Avenue Pump Station (due to the below grade discharge of the existing well system) may be removed if the existing relief well system can discharge at the elevation of the top of the manhole. Additional surface discharge relief wells could be added to provide the required pressure relief due to the higher discharge elevation of the existing wells. However, due to the current assumptions and unknowns on building elevations, additional refinements to the system beyond what is being proposed here are better performed when the required information is available during future design.

Station 254+00 to 275+00

This reach is comprised of a portion of the old "slot" area that appears to have been filled to the elevation of the surrounding ground. The minimum elevation required of the landside blanket is approximately 752 to achieve the minimally acceptable factor of safety with respect to hydraulic gradient of 1.6. Existing topographical information and a field visit have indicated that the landside elevation is currently at or above the minimum required. However, the elevation of the landside blanket should be verified during final design. If the landside elevation is below the minimum required, the area will need to be brought to proper grade to maintain the required factor of safety with respect to hydraulic gradient.

Station 275+00 to 282+00

This reach is comprised of a portion of the "slot" area that remains in its original configuration. Currently the "slot" is landward of the floodwall in this reach. However, portions of the floodwall may be relocated to be landward of the slot in the vicinity of the railroad bridges. In areas where the "slot" will remain landward of the floodwall, the minimum elevation required of the landside blanket is approximately 750 to achieve the minimally acceptable factor of safety with respect to hydraulic gradient of 1.6. This will require filling of the "slot" with impervious material to elevation 750. In locations where the floodwall will be moved landward of the "slot" the slot should be filled with impervious material to the elevation of the riverside fill on the existing floodwall (approximately elevation 748). The existing relief well system should be abandoned in place following all pertinent criteria. The abandonment procedures of the relief wells that have been previously abandoned (southern most 6 wells) should also be verified.

Station 296+00 to 313+00

This reach is characterized by a regionally thin blanket combined with a large low lying area landward of the line of protection between Stations 296+00 and 303+00. The existing relief well system located near the landside toe between Stations 295+00 and 305+00 was designed and constructed to protect the low lying area which contained a large packing plant. The packing plant is no longer present, but some structures remain in use in the low lying area. The calculated factor of safety with respect to hydraulic gradient (with the N500+3 loading) for the area in its current configuration (taking into account the existing relief well system) is approximately 1.0 between Stations 296+00 and 303+00 and approximately 1.2 between Stations 303+00 and 313+00. The existing relief well system would require such significant modification that is not feasible to retain it. To protect the low lying area between Stations 296+00 and 303+00, the existing relief well system should be abandoned in place, and a new system of fully penetrating artesian relief wells should be constructed. A total of 35 fully penetrating artesian relief wells are required between Stations 296+00 and 313+00. Exhibit A-3.21, located in the supplemental exhibits section, shows the proposed locations of the new relief wells, their discharge elevations, and computed individual well flows. A total of 26 fully penetrating artesian relief wells, variably spaced between 25 and 50-ft apart, which discharge at elevation 745 are required to achieve the minimally acceptable factor of safety with respect to hydraulic gradient in the low lying area between Stations 296+00 and 303+00. The total flow from the 26 relief wells was calculated to be approximately 14.5 cfs (or an average of 0.6 cfs per well). The new wells should be placed along the riverward edge of the low lying area, and should discharge directly into the low lying area which will be used as a temporary ponding area. The well system has been designed to achieve the minimally acceptable factor of safety with regards to hydraulic gradient in the low lying area without the consideration of any water being stored in the area. As a result, there are no ponding requirements in the low lying area, i.e. the ponding area can be pumped dry during flood events. Exhibit A-3.22, located in the supplemental exhibits section, shows the computed excess head along the riverward edge of the low lying area, which is the critical area in the reach between Stations 296+00 and 303+00. To protect the area between Stations 303+00 and 313+00, an additional 9 new fully penetrating artesian relief wells should be constructed. The 9 relief wells, variably spaced between 50 and 140-ft apart, should discharge at the landside ground elevation. The total flow from the 9 relief wells was calculated to be approximately 7.5 cfs (or an average of 0.9 cfs per well). The well flow from the 9 relief wells should also be directed to the low lying area which will be used as a temporary ponding area. Exhibit A-3.23, located in the supplemental exhibits section, shows the computed excess head along the landside toe of the line of protection, which is the critical area in the reach between Stations 303+00 and 313+00. The total flow into the ponding area from all relief wells will be approximately 22 cfs.

A-3.8 EXPECTED SETTLEMENT OF DESIGN FEATURES

No calculations were performed to determine the expected settlement of the proposed line of protection raise for the N500+3 condition. This is because no consolidation test data was found to determine the appropriate parameters required for settlement calculations. For feasibility level design the following estimations were made for the proposed raise configurations:

- Floodwall, no settlement
- Cantilever wall on existing levee, minimal settlement
- Earth fill raise on existing levee, 3 inches maximum settlement
- Earth fill in place of existing floodwall, 6 inches maximum settlement

These estimates will be used to determine the overbuild required for the different sections and for quantity estimation. It is recommended that during PED that additional soil sampling and testing be performed so the consolidation characteristics of the foundation materials can be quantitatively determined and settlement analysis performed.

A-3.9 RECOMMENDATIONS FOR PED PHASE

1. Slope Stability
    a. Two reaches, Sta. 228+00 to 245+00 and Sta. 250+60 to 257+65. There is a potential that a T-wall on existing levee section will be used through these reaches. If this section is used through these reaches, the slope stability of the tallest section needs to be analyzed. Currently the tallest existing levee section with T-wall analyzed is 14-ft (section 7). The maximum height in the above listed reaches approaches 18-ft.

    b. Landside levee raise sections. The two levee raise sections analyzed for a landside earth raise were an intermediate height raise and the maximum height raise. The two proposed sections are significantly different, as the shorter section required no stability berms and the taller section required two large stability berms. It is recommended that at least one additional section with a height between the two sections be analyzed in order to minimize the amount of fill and real estate required.

    c. It is recommended that the criteria used for slope stability be evaluated. The existing criteria were used for the Feasibility level design, however due to Hurricane Katrina there was a lot of discussion about increasing factors of safety during this time, and some interim guidance had been published. By PED the criteria for flood risk management projects may be revised.

2. Underseepage

    a. It is recommended that all building elevations be confirmed, and proposed layouts of remedial measures are refined accordingly.

b. It is recommended that the levee unit be revisited for additional features, such as pits and low spots which need special attention with respect to the underseepage analysis.

c. It is recommended that all existing relief wells to remain in use be pump tested and inspected to ensure required well flows can be achieved and adequate condition of the wells.

d. It is recommended that any changes in Corps of Engineers (or local district) guidance which governs underseepage analysis methods or criteria be captured during final design.

3. It is recommended that a drilling and testing program be implemented to verify gaps in the existing data and to meet all criteria regarding sub-surface investigation intensity. Those include (at a minimum):

   a. Landside blanket thickness between 246+00 and 254+00.

   b. Soil Strength Testing

      i. Undrained strengths for the fill material and the blanket materials both under the existing levee sections and in the natural blanket outside the levee footprint.

      ii. R-bar triaxial testing on the fill section and the natural blanket materials to develop strength parameters needed for rapid drawdown analysis.

      iii. Consolidation testing in reaches to receive fill for purposes of settlement estimation.

4. Recommend a full topographic survey in the critical zone of the line of protection, including all the way to the riverbank.

5. Attempt to provide unrestricted vehicle access along the entire length of the line of protection for inspection. Currently only small reaches of the protection can be inspected at one time, and access to adjacent reaches requires navigating around the industries being protected.

6. Recommend evaluating the impact of ground discharging relief wells on the interior drainage. The quantity of expected discharge from proposed wells for the N500+3 conditions would indicate that interior flooding could be a significant problem.

7. A ground water study should take place in the area of the proposed cutoff wall to ensure local water interests will not be affected.

A-3.10 REFERENCES

1. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Argentine Unit, Volume I, Dated 1979.

2. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Argentine Unit, Volume I, Appendix II, Dated 1951 - 1974.

3. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Armourdale Unit, Volume I, Dated 1979.

4. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Armourdale Unit, Volume I, Appendix I, Dated 1951 - 1954.

5. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Armourdale Unit, Volume II, Appendix I, Dated 1954 - 1976.

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7. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial Unit Kansas Section, Volume I, Appendix I, Dated 1950 - 1955.

8. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial Unit Kansas Section, Volume II, Appendix I, Dated 1979.

9. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial Unit Missouri Section, Volume I, Dated 1981.

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13. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Fairfax-Jersey Creek Unit, Volume I, Dated 1979.

14. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Fairfax-Jersey Creek Unit, Volume I, Appendix I, Dated 1944 - 1955.

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18. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, North Kansas City Unit Lower Section, Volume I, Appendix I, Dated 1948 - 1979.

19. Kansas Citys Flood Control Project, Definite Project Report on North Kansas City Unit Unit, Supplement on Interior Drainage, Dated 1951.

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21. Baecher, G. B., & J. J. Christian (2000), "Uncertainty, Probability, and Geotechnical Data," paper presented at Performance Confirmation of Constructed Geotechnical Facilities, ASCE, Amherst, MA; April 9-12.

22. Hunt, R. E., (1986), *Geotechnical Engineering Analysis and Evaluation,* New York: McGraw-Hill Book Company.

23. Reese, L. C., Wang, S. T., & Arrellaga, J., (1998), "Computer Program APILE Plus – A Program for the Analysis of the Axial Capacity of Driven Piles" ENSOFT, INC., Austin, TX.

24. US Army Corps of Engineers (1999), *Reconnaissance Report – Kansas Citys, Missouri and Kansas Flood Damage Reduction Project,* Kansas City District.

25. Wolff, T. F., (1985), "Analysis and Design of Embankment Dam Slopes: A Probabilistic Approach", Doctoral Dissertation presented to Purdue University, West Lafayette, IND.

26. Wright, S. G., (1999), "UTEXAS 4 – A Computer Program for Slope Stability Calculations", prepared for the Department of the Army, U. S. Army Corps of Engineers, Washington D. C.

27. US Army Corps of Engineers (May 1971), *Design Memorandum No 3 – Armourdale Unit*, Kansas City District.

A-3.11 SUPPLEMENTAL EXHIBITS

Exhibit A-3.1
Exiting Conditions Underseepage Analysis

Armourdale Levee Kansas City Phase 2 Feasibility
Thursday, December 28, 2006
Levee Foundation Information - EXISTING CONDITIONS, Water to Top of Levee

Blanket Unit Weight = 115.0 pcf (saturated)

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio River Side k_f/k_{br}	Permeability Ratio Land Side k_f/k_{bl}	Impervious Blanket Thickness (ft) River Side z_{br}	Impervious Blanket Thickness (ft) Levee z_{bc}	Impervious Blanket Thickness (ft) Land Side z_{bl}	Pervious Blanket Thickness (ft) d	Seepage Length (ft) River Side L_1	Seepage Length (ft) Levee L_2	Seepage Length (ft) Land Side L_3	Factor c_r	Factor c_l	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_0	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Factor of Safety for Piping (i_c/i_o)	Remarks
5+00 UE to 30+00	774.0	746	770	735	735	671	4.0	300	300	11	23	35	64	260	100	1000	0.002176	0.001220	235	920	2.84	0.08	0.84	10.39	OK
30+00 to 47+00	773.5	746	760	730	730	671	13.5	300	300	16	23	30	58	200	100	1000	0.001879	0.001372	191	729	9.65	0.32	0.84	2.62	OK
47+00 to 57+00	773.3	750	762	740	740	670	11.3	300	300	10	16	22	70	200	100	1000	0.002182	0.001471	188	680	7.94	0.36	0.84	2.34	OK
57+00 to 62+00	773.0	750	762	735	735	670	11.0	300	300	15	21	27	65	350	100	1000	0.001849	0.001376	308	726	7.94	0.26	0.84	3.23	OK
62+00 to 66+00	773.3	750	762	740	740	671	11.3	400	400	10	16	22	69	300	50	1000	0.001903	0.001283	271	779	8.90	0.36	0.84	2.32	OK
66+00 to 73+00	773.0	755	760	748	748	671	13.0	300	300	7	9.5	12	77	175	50	1000	0.002487	0.001898	165	526	9.23	0.77	0.84	1.10	FS below current criteria
73+00 to 79+00	772.5	753	760	732	746	671	12.5	300	300	21	18	15	61	75	50	1000	0.001813	0.001909	75	524	10.10	0.67	0.84	1.25	FS below current criteria
79+00 to 86+00	772.0	750	760	730	730	667	12.0	400	400	20	25	30	63	100	75	1000	0.001409	0.001150	99	868	10.90	0.33	0.84	2.53	OK
86+00 to 93+00	772.0	760	760	735	735	667	12.0	400	400	15	20	25	68	75	100	1000	0.001566	0.001213	75	825	9.90	0.40	0.84	2.13	OK
93+00 to 100+00	771.0	745	760	730	730	667	11.0	250	250	15	22.5	30	63	75	125	1000	0.002057	0.001456	74	687	8.53	0.28	0.84	2.97	OK
100+00 to 118+00	770.5	742	758	732	732	667	12.5	350	350	10	18	26	65	150	60	1000	0.002097	0.001500	145	759	9.87	0.38	0.84	2.22	sand layer between ground surface and 5 feet below ground surface not considered in analysis. Restoration drawings indicate 5' minimum impervious layer on riverside to el 735 to cut off seepage through sand layer. OK
118+00 to 130+00	770.0	744	756	733	733	667	14.0	350	350	11	17	23	68	250	125	1000	0.001884	0.001372	231	729	8.40	0.41	0.84	2.06	OK
130+00 to 175+00	769.0	742	760	730	730	672	9.0	300	300	12	21	30	58	225	100	1000	0.002189	0.001384	206	722	6.31	0.21	0.84	4.01	sand lenses between ground surface and 755ish not considered as seepage path. Restoration drawings indicate 5' minimum impervious layer on riverside to el 740 to cut off seepage through sand lenses. OK
175+00 to 186+00	768.0	742	762	725	725	656	6.0	300	300	17	27	37	70	130	100	1000	0.001674	0.001134	128	891	4.77	0.13	0.84	6.54	OK
186+00 to 195+00	767.5	740	752	728	728	668	6.5	300	300	12	18	24	60	80	100	1000	0.002152	0.001521	79	657	6.68	0.28	0.84	3.03	existing relief wells and buildings not considered in analysis. there is apparent sand pockets on the landside, but were ignored in analysis as rehab drawings indicate impervious fill on riverside to elevation 740ish. additional borings may be needed.
195+00 to 205+00	767.0	740	752	728	728	668	10.0	300	300	12	18	24	60	80	100	1000	0.002152	0.001521	79	657	7.86	0.33	0.84	2.57	existing relief wells and buildings not considered in analysis. there is apparent sand pockets on the landside below the extent of the impervious fill on the river side at 745. for the sake of this analysis, it is assumed they are not continuous under the levee. additional borings may be needed.
205+00 to 215+00	766.5	740	752	728	728	668	14.5	300	300	12	18	24	60	80	100	1000	0.002152	0.001521	79	657	11.39	0.47	0.84	1.79	existing relief wells and buildings not considered in analysis. there is apparent sand pockets on the landside, but were ignored in analysis as rehab drawings indicate impervious fill on riverside to elevation 742ish. additional borings may be needed
215+00 to 225+00	766.0	740	748	722	722	668	18.0	300	300	18	22	26	54	80	100	1000	0.001852	0.001541	79	649	14.10	0.54	0.84	1.55	existing relief wells and buildings not considered in analysis.
225+00 to 235+00	765.5	740	748	725	730	668	17.5	300	300	15	16.5	18	57	80	100	1000	0.001974	0.001802	79	559	13.22	0.73	0.84	1.15	existing relief wells and buildings not considered in analysis.
235+00 to 245+00	765.0	740	749	739	730	668	16.0	300	300	10	14.5	19	62	80	100	1000	0.002319	0.001682	79	594	12.30	0.65	0.84	1.30	existing relief wells and buildings not considered in analysis.
245+00 to 251+00	764.5	739	751	730	730	663	13.5	200	200	8	14.5	21	77	75	95	1000	0.002849	0.001758	74	569	10.41	0.50	0.84	1.70	existing relief wells and buildings not considered in analysis
251+00 to 254+00	763.5	741	752	740	740	663	11.5	350	350	1	6.5	12	77	100	125	1000	0.006901	0.001759	89	569	9.35	0.70	0.84	1.21	blanket appears to thin out here based on riverside borings. additional borings on the land side may indicate a thicker blanket on the land side. if this is the case, underseepage might not be a problem. FS below current criteria.
254+00 to 265+00	763.2	744	752	729	729	671	11.2	400	400	16	20	24	57	125	50	1000	0.001659	0.001352	121	740	6.67	0.38	0.84	2.23	analysis does not include shallow sand lenses, as they have been cut off as indicated by rehab drawings showing impervious fill down to 740 on river side. field investigation indicated slot area is filled to 752
265+00 to 282+00	762.5	742	750	725	725	678	12.5	350	300	17	21	25	47	100	50	1000	0.002843	0.001694	89	594	10.00	0.40	0.84	2.11	existing wells not considered in analysis. Slot not considered in blanket theory calcs. HGL = 760. Slot El = 739. h_t in slot = 760-739-5 (per DM 3) = 16. $Z_{(slot)}$ = 739-725 = 14. $i_{(c)}$ = 1.14. **FS_{slot} = 0.74 w/o wells. with wells - FS = 0.9**
282+00 to 286+00	762.0	739	746	717	717	678	16.0	400	400	21	25	29	39	75	75	1000	0.001747	0.001487	75	673	13.59	0.46	0.84	1.87	OK
286+00 to 296+00	761.7	739	750	726	726	678	11.7	400	400	13	19	25	47	100	50	1000	0.002023	0.001492	88	686	9.62	0.39	0.84	2.18	OK
296+00 to 303+00	761.5	740	742	728	728	678	19.5	400	400	12	13	14	50	75	75	1000	0.002041	0.001890	74	529	15.21	1.09	0.84	0.78	existing wells not considered in analysis. **FS = 1.0 - 1.4 with wells.** below current criteria
303+00 to 313+00	760.5	740	747	730	730	678	13.5	400	400	10	13.5	17	52	50	100	1000	0.002193	0.001682	50	595	10.78	0.63	0.84	1.33	FS below current criteria
313+00 to 40+00LE	760.5	745	750	725	725	700	10.5	400	400	20	22.5	25	25	110	100	1000	0.002238	0.002000	105	600	7.42	0.30	0.84	2.84	sand layers are located on riverside and landside between elevations 738ish and ground surface. Restoration drawings indicate 5' minimum impervious layer on riverside to el 740 to cut off seepage through sand layer. performance during 93 in this area indicates the layers do not pose a problem. OK
40+00 LE to 59+60LE	760.0	745	758	725	725	700	2.0	400	400	20	26.5	33	25	110	100	1000	0.002236	0.001741	106	574	1.47	0.64	0.84	18.94	sand layers are located on riverside and landside between elevations 740ish and ground surface. Restoration drawings indicate 5' minimum impervious layer on riverside to el 740 to cut off seepage through sand layer. performance during 93 in this area indicates the layers do not pose a problem. OK

EXHIBIT A-3.2
Typical shape of the normal probability distribution function showing the expected value or mean, E[X]

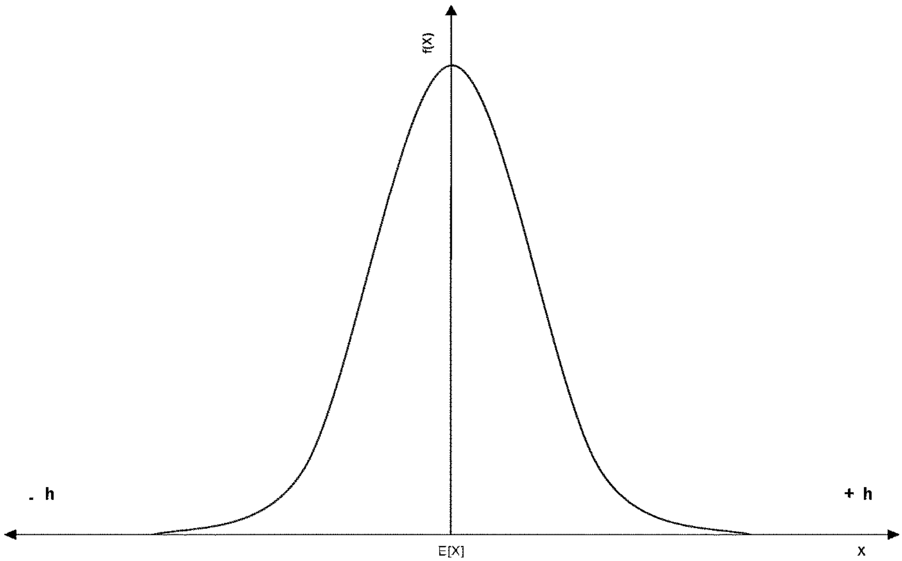


EXHIBIT A-3.3
Typical shape of the log-normal distribution function showing the expected value, E[X]

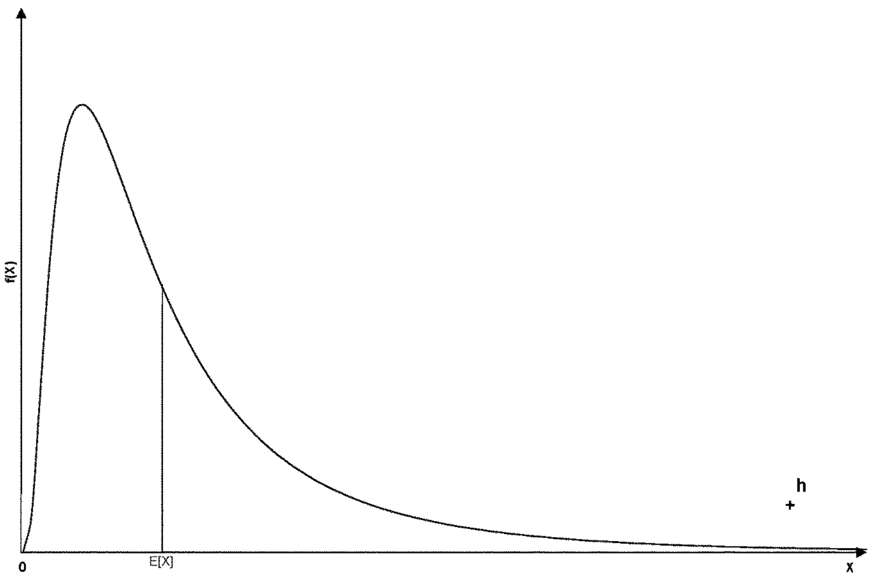


EXHIBIT A-3.4
Hypothetical normal probability distribution showing the probabilistic parameters

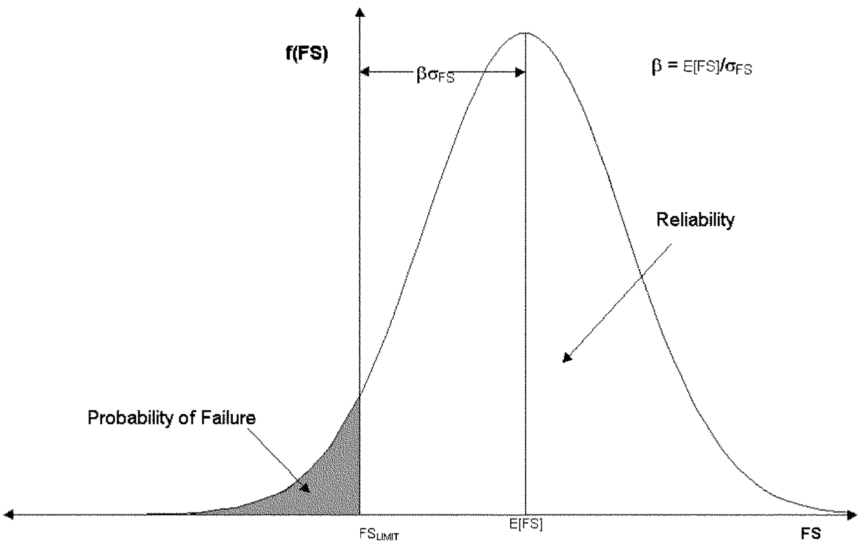


EXHIBIT A-3.5
Normal probability distribution for the natural log of the factor of safety, assuming that the factor of safety is log-normally distributed

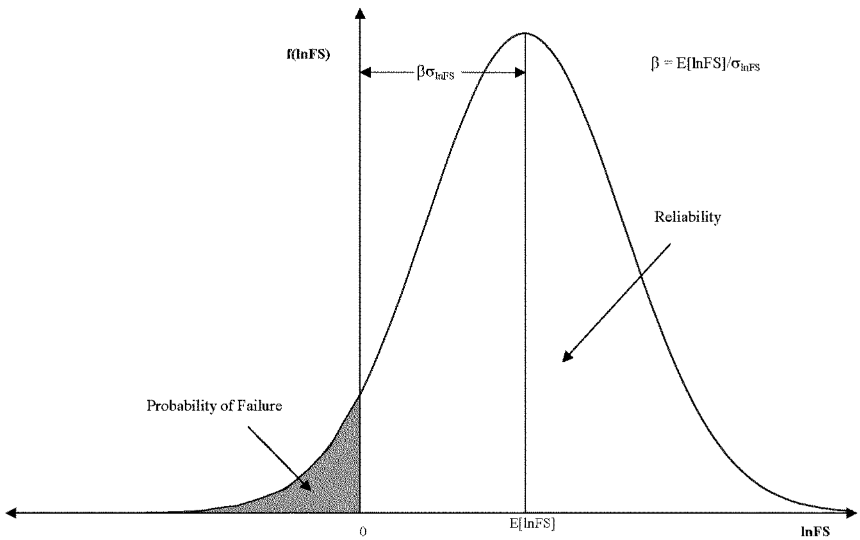


EXHIBIT A-3.6
Normal probability distribution for the natural log of the hydraulic gradient, assuming that the hydraulic gradient is log-normally distributed where the failure gradient is defining the limit state

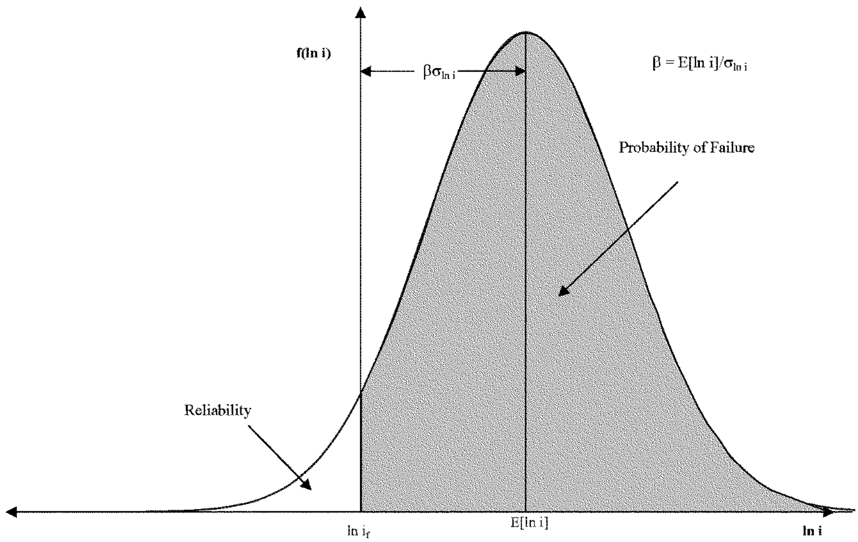


EXHIBIT A-3.7
The probability distribution curve illustrating the assumptions used in developing the Taylor Series Approximation

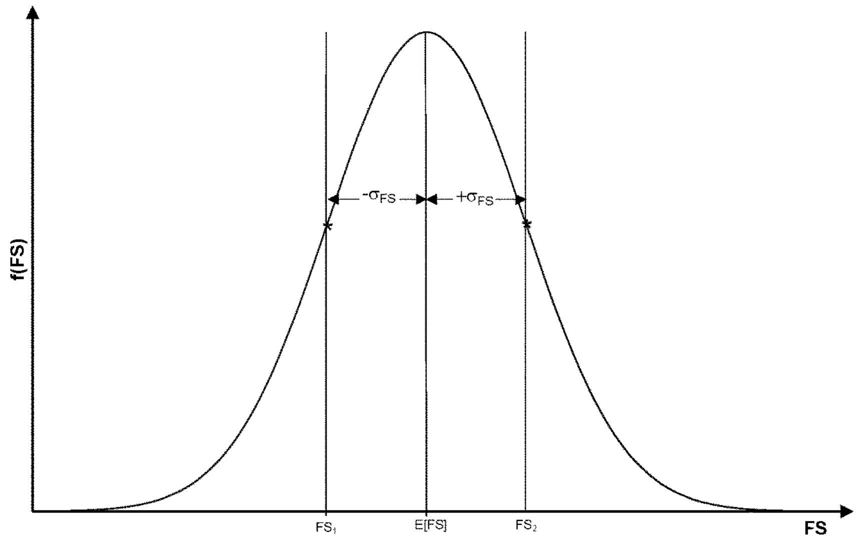


EXHIBIT A-3.8
Expected Value of Factor of Safety and Probability of Failure Relation

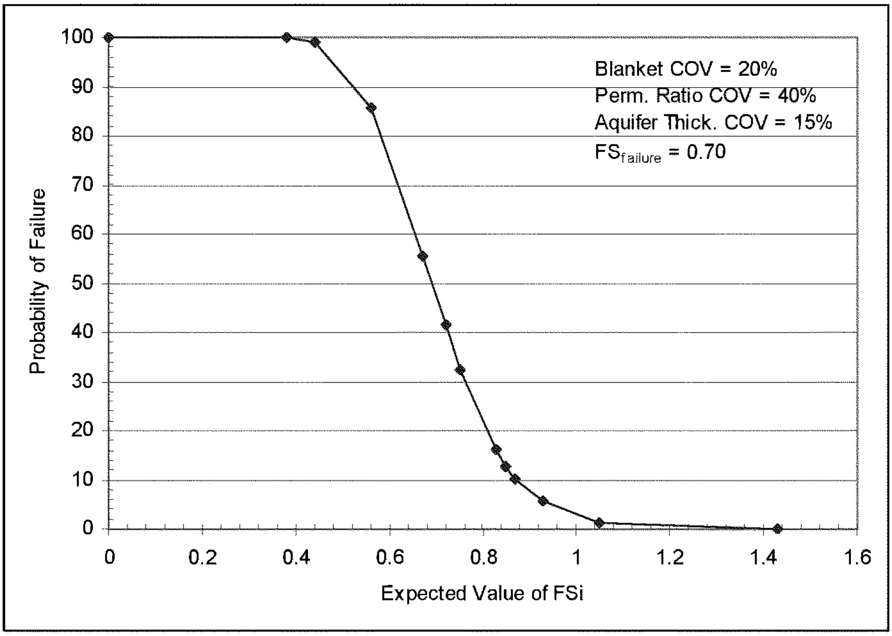


EXHIBIT A-3.9
Probability of Failure Due to Underseepage Failure at Station 276+00

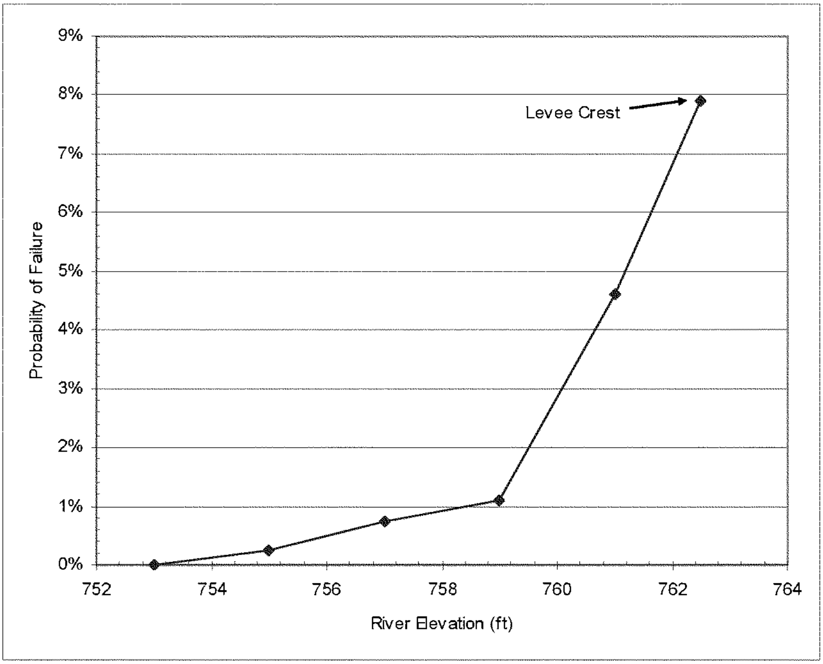


EXHIBIT A-3.10
Probability of Failure Due to Stability Failure at Station 222+00

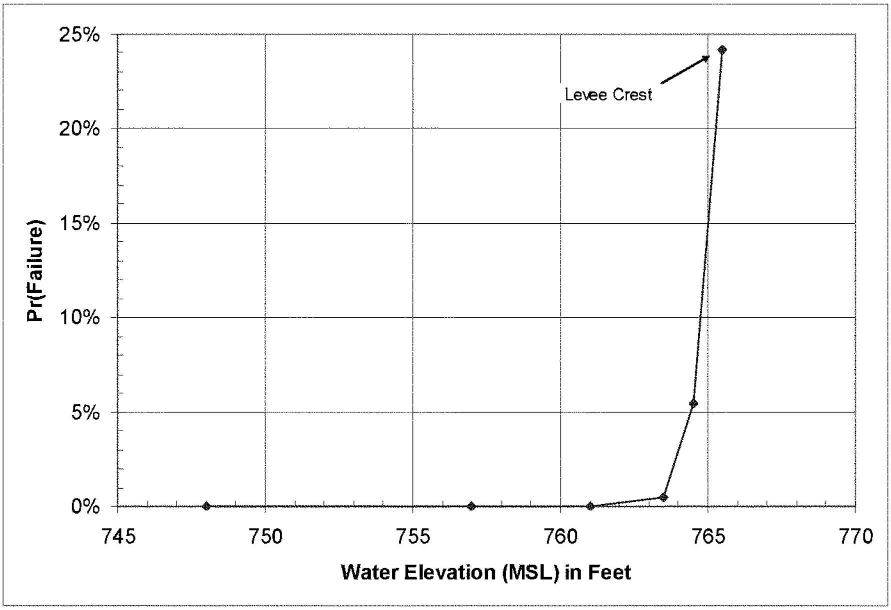


EXHIBIT A-3.11

N500+3 Stability Analysis Calculations – Armourdale Unit

```
HEADING
    Kansas City Levees - Armourdale Unit
    Station 7+00UE Stability Analysis
    End of Construction Analysis - Riverside Fill
    March 2007

PROFILE LINES
         3    1 Levee, moist
              256.08    774.00
              263.00    774.00
              290.00    765.00

         2    2 Levee, Raise, saturated
              110.00    750.00
              241.00    778.00
              256.08    774.00

         4    2 Levee saturated
              199.00    756.00
              253.00    774.00
              256.08    774.00
              290.00    765.00

         5    4 Foundation, Silt, saturated
                 .00    750.00
              110.00    750.00
              199.00    756.00
              290.00    765.00
              500.00    765.00

         6    5 Foundation, Sand, saturated
                 .00    735.00
              500.00    735.00

         7    6 Bedrock
                 .00    670.00
              500.00    670.00

         1    1 Levee Raise, moist
              241.00    778.00
              251.00    778.00
              263.00    774.00

         8    1 Berm
              272.00    771.00
              284.00    771.00
              304.50    765.00


MATERIAL PROPERTIES
     1 Levee, moist
          115.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     2 Levee, saturated
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     4 Foundation, Silt, saturated
```

```
        115.00 Unit Weight
        Conventional Shear
            500.00        .00
        No Pore Pressure
   5 Foundation, Sand, saturated
        120.00 Unit Weight
        Conventional Shear
              .00     32.00
        Piezometric Line
        2
   6 Bedrock
        150.00 Unit Weight
        Very Strong

PIEZOMETRIC LINES
        2      62.40 Foundation Sand Pressures
                 .00     750.00
              143.00     750.00
              199.00     756.00
              290.00     765.00
              500.00     765.00


INTERPOLATION DATA
Pore Water Pressure
```

.00	750.00	1747.20	4
.00	735.00	2683.20	4
40.00	750.00	1747.20	4
40.00	735.00	2652.00	4
80.00	750.00	1747.20	4
80.00	735.00	2620.80	4
110.00	750.00	1747.20	4
110.00	735.00	2597.10	4
143.00	750.00	1747.20	4
143.00	735.00	2570.90	4
170.00	752.89	1538.70	4
170.00	744.00	2040.70	4
170.00	735.00	2549.70	4
199.00	756.00	1322.50	4
199.00	746.00	1[illegible]96.20	4
199.00	735.00	25[illegible]7.[illegible]0	4
220.00	758.08	11[illegible]7.80	4
220.00	735.00	2510.30	4
220.00	746.00	1875.20	4
241.00	760.15	10[illegible].10	4
241.00	748.00	[illegible]741.40	4
241.00	735.00	2496.00	4
266.00	[illegible]	594.60	4
266.00	7[illegible]8.00	1589.80	4
266.00	735.00	2474.20	4
290.00	765.00	.00	4
290.00	750.00	1227.70	4
290.00	735.00	2455.40	4
330.00	765.00	.00	4
330.00	750.00	1212.10	4
330.00	735.00	2424.20	4
370.00	765.00	.00	4
370.00	750.00	1198.10	4
370.00	735.00	2396.20	4
410.00	765.00	.00	4
410.00	750.00	1184.10	4

NOT USED FOR THIS ANALYSIS

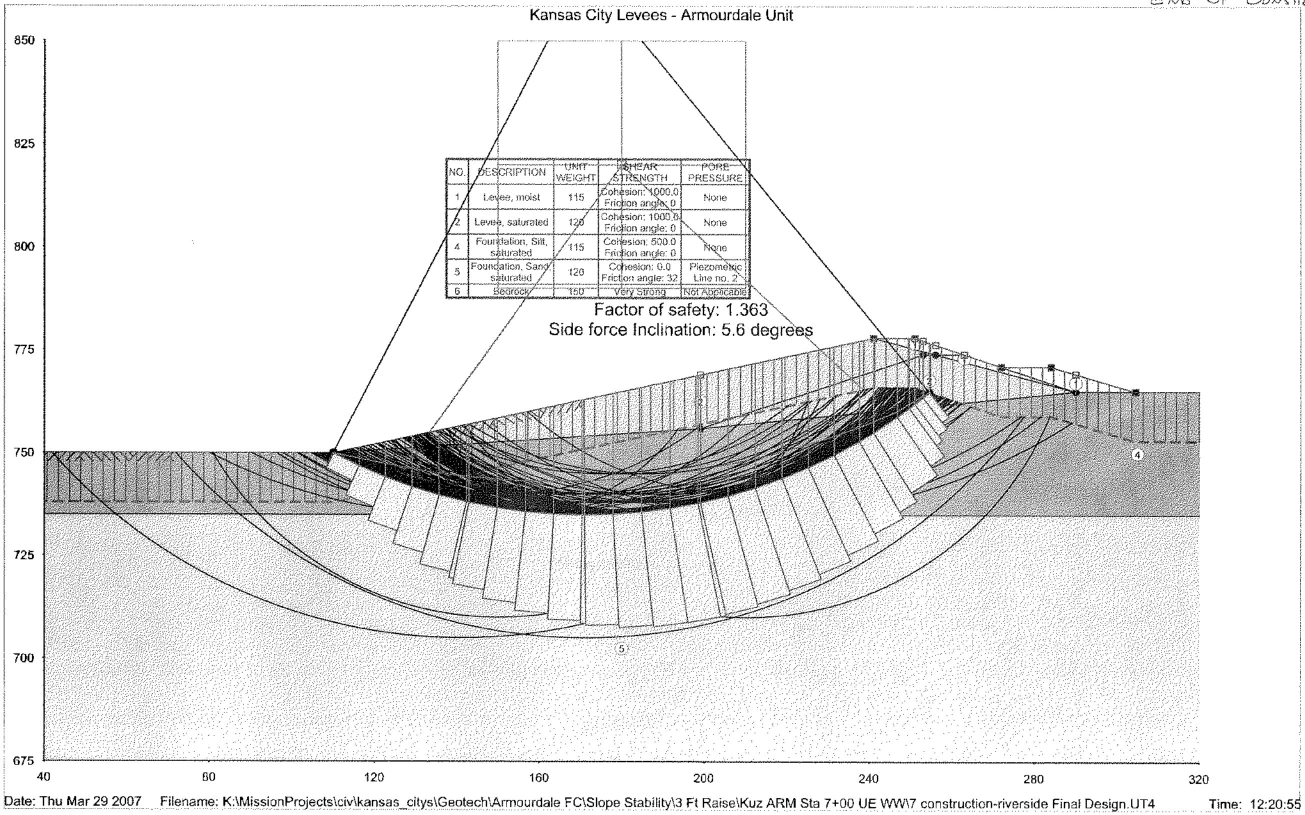
```
         410.00     735.00    2368.10          4
         450.00     765.00        .00          4
         450.00     750.00    1171.60          4
         450.00     735.00    2343.10          4
         500.00     765.00        .00          4
         500.00     750.00    1161.90          4
         500.00     735.00    2323.80          4

ANALYSIS/COMPUTATION
     Circular Search 1
         180.00     820.00       1.00     670.00
     Radius
          80.00
SINgle-stage Computations
TRIal max
 100
SAVe n most
 150
LONg-form output
CRAck
          12.00          D
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

29 Mar 2007 (1)

End of Construction



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HEADING
    Kansas City Levees - Armourdale Unit
    Station 7+00UE Stability Analysis
    Steady Seepage Analysis With Landside Berm - Riverside Fill
    March 2007

PROFILE LINES
         3    1 Levee, moist
              256.08     774.00
              263.00     774.00
              290.00     765.00

         2    2 Levee, Raise, saturated
              110.00     750.00
              241.00     778.00
              256.08     774.00

         4    2 Levee saturated
              199.00     756.00
              253.00     774.00
              256.08     774.00
              290.00     765.00

         5    4 Foundation, Silt, saturated
                 .00     750.00
              110.00     750.00
              199.00     756.00
              290.00     765.00
              500.00     765.00

         6    5 Foundation, Sand, saturated
                 .00     735.00
              500.00     735.00

         7    6 Bedrock
                 .00     670.00
              500.00     670.00

         1    1 Levee Raise, moist
              241.00     778.00
              251.00     778.00
              263.00     774.00

         8    1 Berm
              272.00     771.00
              284.00     771.00
              304.50     765.00


MATERIAL PROPERTIES
     1 Levee, moist
          115.00 Unit Weight
          Conventional Shear
                 .00      29.00
          No Pore Pressure
     2 Levee, saturated
          120.00 Unit Weight
          Conventional Shear
                 .00      29.00
          Piezometric Line
          1
```

```
    4 Foundation, Silt, saturated
        115.00 Unit Weight
        Conventional Shear
               .00     26.00
        Interpolate Pore Water Pressure
    5 Foundation, Sand, saturated
        120.00 Unit Weight
        Conventional Shear
               .00     32.00
        Piezometric Line
        2
    6 Bedrock
        150.00 Unit Weight
        Very Strong

PIEZOMETRIC LINES
        1      62.40 1 - Line of Seepage
                 .00     778.00
              241.00     778.00
              256.08     774.00
              290.00     765.00
              500.00     765.00

        2      62.40 2 - Foundation Sand Pressures
                 .00     778.00
              290.00     774.35
              370.00     773.40
              500.00     772.04


DISTRIBUTED LOADS
     1
INTERPOLATION DATA
Pore Water Pressure
             .00     750.00    1747.20          4
             .00     735.00    2683.20          4
           40.00     750.00    1747.20          4
           40.00     735.00    2652.00          4
           80.00     750.00    1747.20          4
           80.00     735.00    2620.80          4
          110.00     750.00    1747.20          4
          110.00     735.00    2597.10          4
          143.00     750.00    1747.20          4
          143.00     735.00    2570.90          4
          170.00     752.89    1538.70          4
          170.00     744.00    2040.70          4
          170.00     735.00    2549.70          4
          199.00     756.00    1322.50          4
          199.00     746.00    1896.20          4
          199.00     735.00    2527.20          4
          220.00     758.08    1177.80          4
          220.00     735.00    2510.30          4
          220.00     746.00    1875.20          4
          241.00     760.15    1036.10          4
          241.00     748.00    1741.40          4
          241.00     735.00    2496.00          4
          266.00     762.63     594.60          4
          266.00     748.00    1589.80          4
          266.00     735.00    2474.20          4
          290.00     765.00        .00          4
          290.00     750.00    1227.70          4
```

```
        290.00     735.00    2455.40          4
        330.00     765.00        .00          4
        330.00     750.00    1212.10          4
        330.00     735.00    2424.20          4
        370.00     765.00        .00          4
        370.00     750.00    1198.10          4
        370.00     735.00    2396.20          4
        410.00     765.00        .00          4
        410.00     750.00    1184.10          4
        410.00     735.00    2368.10          4
        450.00     765.00        .00          4
        450.00     750.00    1171.60          4
        450.00     735.00    2343.10          4
        500.00     765.00        .00          4
        500.00     750.00    1161.90          4
        500.00     735.00    2323.80          4

ANALYSIS/COMPUTATION
     Circular Search 1
        290.00     810.00       1.00     670.00
     Radius
         55.00
SINgle-stage Computations
RIGht Face of Slope
SAVe n most
 150
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
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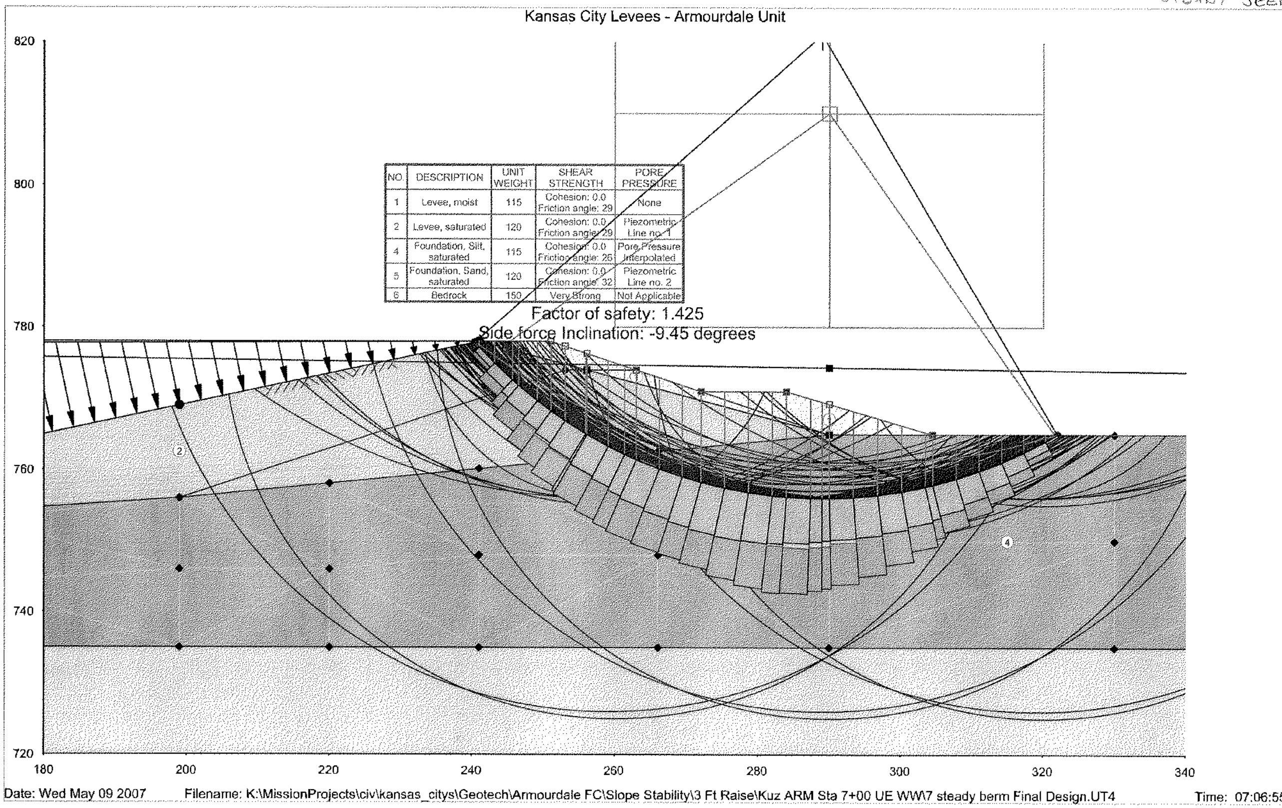
29 MAR 2007

STEADY SEEPAGE



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee, moist	115	Cohesion: 0.0 Friction angle: 29	None
2	Levee, saturated	120	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
4	Foundation, Silt, saturated	115	Cohesion: 0.0 Friction angle: 25	Pore Pressure Interpolated
5	Foundation, Sand, saturated	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 2
6	Bedrock	150	Very Strong	Not Applicable

```
HEADING
    KC Levees Phase 2
    Armourdale Levee Unit
    Station 60+00 to 75+00
    Levee Raise to Replace Existing Floodwall
    End of Construction Case

PROFILE LINES
         1    1 Bedrock
                 .00         .00
              373.20         .00

         2    2 Foundation Sand
                 .00       75.00
              100.00       75.00
              225.00       55.00
              373.20       55.00

         3    3 Foundation Clays and Silts
                 .00       90.00
              101.20       90.00
              226.20       70.00
              246.20       70.00
              373.20       70.00

         4    4 Old Levee Impervious Fill
              101.20       90.00
              176.20       90.00
              246.20       70.00

         5    5 New Levee Impervious Fill
              101.20       90.00
              152.20      107.00
              162.20      107.00
              207.20       92.00
              273.20       70.00

         6    6 New Levee - Berm - Impervious Fill
              207.20       92.00
              257.20       92.00
              299.20       78.00
              323.20       70.00

         7    7 New Levee - Berm2 - Impervious Fill
              299.20       78.00
              324.20       78.00
              348.20       70.00


MATERIAL PROPERTIES
     1 Bedrock
          150.00 Unit Weight
          Very Strong
     2 Foundation Sand
          120.00 Unit Weight
          Conventional Shear
               .00     32.00
          Piezometric Line
          1
     3 Foundation Clays and Silts
          115.00 Unit Weight
```

```
          Conventional Shear
              500.00        .00
          No Pore Pressure
     4 Old Levee Impervious Fill
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     5 New Levee Impervious Fill
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     6 New Levee - Berm - Impervious Fill
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     7 New Levee - Berm2 - Impervious Fill
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure

PIEZOMETRIC LINES
         1      62.40 Groundwater
                  .00      90.00
               101.20      90.00
               226.20      70.00
               373.20      70.00


SLOPE GEOMETRY

ANALYSIS/COMPUTATION
     Circular Search 1
         265.00    225.00        .10        .00        .00
     Radius
         175.00
SINgle-stage Computations
RIGht Face of Slope
FACtor of safety
 1.5
ITEration limit
 1000
SAVe n most
 5000
LONg-form output
CRAck
          14.00         D
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
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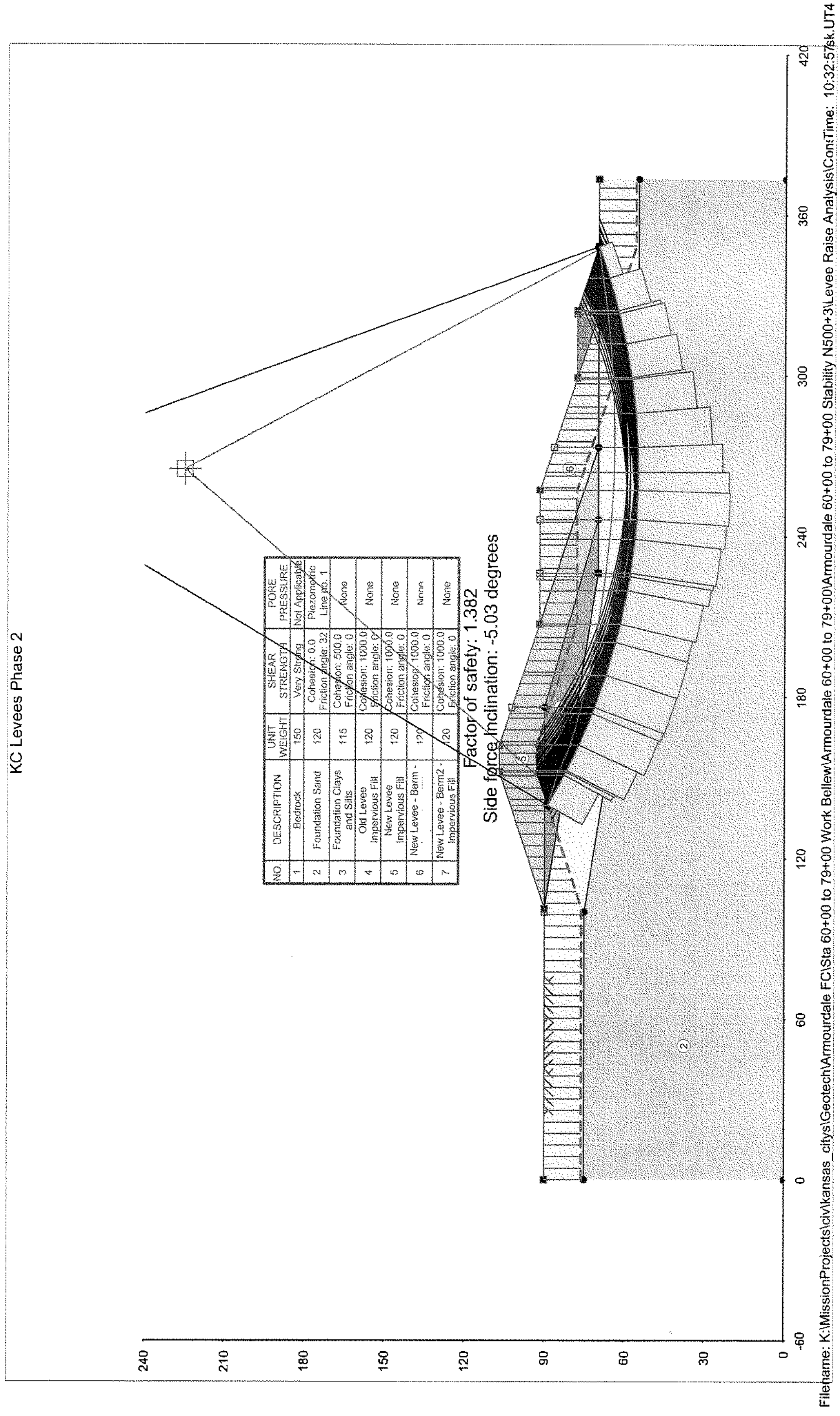
KC Levees Phase 2
NO. | DESCRIPTION | UNIT WEIGHT | SHEAR STRENGTH | PORE PRESSURE
1 | Bedrock | 150 | Very Strong | Not Applicable
2 | Foundation Sand | 120 | Cohesion: 0.0 Friction angle: 32 | Piezometric Line no. 1
3 | Foundation Clays and Silts | 115 | Cohesion: 500.0 Friction angle: 0 | None
4 | Old Levee Impervious Fill | 120 | Cohesion: 1000.0 Friction angle: 0 | None
5 | New Levee Impervious Fill | 120 | Cohesion: 1000.0 Friction angle: 0 | None
6 | New Levee - Berm - | 120 | Cohesion: 1000.0 Friction angle: 0 | None
7 | New Levee - Berm2 - Impervious Fill | 120 | Cohesion: 1000.0 Friction angle: 0 | None
Factor of safety: 1.382
Side force Inclination: -5.03 degrees
240
210
180
150
120
90
60
30
0
-60
0
60
120
180
240
300
360
420
Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Sta 60+00 to 79+00 Work Bellew\Armourdale 60+00 to 79+00\Armourdale 60+00 to 79+00 Stability N500+3\Levee Raise Analysis\ConTime: 10:32:57sk.UT4

```
HEADING
    KC Levees Phase 2
    Armourdale Levee Unit
    Station 60+00 to 75+00
    Levee Raise to Replace Existing Floodwall
    Steady State Seepage Case - Full Head

PROFILE LINES
        1     1 Bedrock
                 .00        .00
              373.20        .00

        2     2 Foundation Sand
                 .00      75.00
              100.00      75.00
              225.00      55.00
              373.20      55.00

        3     3 Foundation Clays and Silts
                 .00      90.00
               76.20      90.00
              101.20      90.00
              226.20      70.00
              246.20      70.00
              373.20      70.00

        4     4 Old Levee Impervious Fill
              101.20      90.00
              176.20      90.00
              246.20      70.00

        5     5 New Levee Impervious Fill
              101.20      90.00
              116.20      95.00
              152.20     107.00
              162.20     107.00
              207.20      92.00
              273.20      70.00

        6     6 New Levee - Berm - Impervious Fill
              207.20      92.00
              257.20      92.00
              299.20      78.00
              323.20      70.00

        7     7 New Levee - Berm2 - Impervious Fill
              299.20      78.00
              324.20      78.00
              348.20      70.00


MATERIAL PROPERTIES
      1 Bedrock
           150.00 Unit Weight
           Very Strong
      2 Foundation Sand
           120.00 Unit Weight
           Conventional Shear
                 .00      32.00
           Piezometric Line
           2
```

```
     3 Foundation Clays and Silts
          115.00 Unit Weight
          Conventional Shear
               25.00     26.00
          Interpolate Pore Water Pressure
     4 Old Levee Impervious Fill
          120.00 Unit Weight
          Conventional Shear
               25.00     29.00
          Piezometric Line
          1
     5 New Levee Impervious Fill
          120.00 Unit Weight
          Conventional Shear
               25.00     29.00
          Piezometric Line
          1
     6 New Levee - Berm - Impervious Fill
          120.00 Unit Weight
          Conventional Shear
               25.00     29.00
          Piezometric Line
          1
     7 New Levee - Berm2 - Impervious Fill
          120.00 Unit Weight
          Conventional Shear
               25.00     29.00
          Piezometric Line
          1

PIEZOMETRIC LINES
         1       62.40 Conservative Line of Seepage
                   .00       90.00
                 76.20       90.00
                101.20       90.00
                140.00       90.00
                162.20      107.00
                373.20      107.00

         2       62.40 Foundation Sand Pressures
                   .00       98.00
                162.00       98.00
                162.20      107.00
                373.20      107.00


SLOPE GEOMETRY

DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
            .00      90.00        .00          3
            .00      75.00    1465.00          3
          25.00      90.00        .00          3
          25.00      75.00    1435.00          3
          50.00      90.00        .00          3
          50.00      75.00    1465.00          3
          75.00      90.00        .00          3
          75.00      75.00    1435.00          3
         100.00      75.00    1435.00          3
```

```
          101.20      90.00         .00          3
          111.20      73.20     1548.00          3
          111.20      88.40      100.00          3
          121.20      71.60     1647.00          3
          121.20      86.80      200.00          3
          131.20      70.00     1747.00          3
          131.20      85.20      300.00          3
          141.20      68.40     1847.00          3
          141.20      83.60      399.00          3
          151.20      66.80     2053.00          3
          151.20      82.00     1104.00          3
          161.20      65.20     2565.00          3
          161.20      80.40     1616.00          3
          162.20      65.00     2650.00          3
          162.20      80.00     1714.00          3
          165.20      64.60     2650.00          3
          165.20      79.60     1714.00          3
          172.20      63.40     2720.00          3
          172.20      78.60     1772.00          3
          182.20      61.80     2820.00          3
          182.20      77.00     1872.00          3
          192.20      60.20     2920.00          3
          192.20      75.40     1972.00          3
          202.20      58.60     3020.00          3
          202.20      73.80     2072.00          3
          212.20      57.00     3120.00          3
          212.20      72.20     2172.00          3
          222.20      55.40     3220.00          3
          222.20      70.60     2271.00          3
          225.00      55.00     3245.00          3
          225.00      70.00     2310.00          3
          250.00      55.00     3245.00          3
          250.00      70.00     2310.00          3
          275.00      55.00     3245.00          3
          275.00      70.00     2310.00          3
          300.00      55.00     3245.00          3
          300.00      70.00     2310.00          3
          325.00      55.00     3245.00          3
          325.00      70.00     2310.00          3
          350.00      55.00     3245.00          3
          350.00      70.00     2310.00          3
          373.20      55.00     3245.00          3
          373.20      70.00     2310.00          3


ANALYSIS/COMPUTATION
     Circular Search 1
          80.00     250.00          .10          .00          .00
     Tangent
          75.00
SINgle-stage Computations
LEFt Face of Slope
FACtor of safety
 1.5
ITEration limit
 2500
SAVe n most
 5000
CHAnge initial trial factor of safety
LONg-form output
SORt radii
```

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CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
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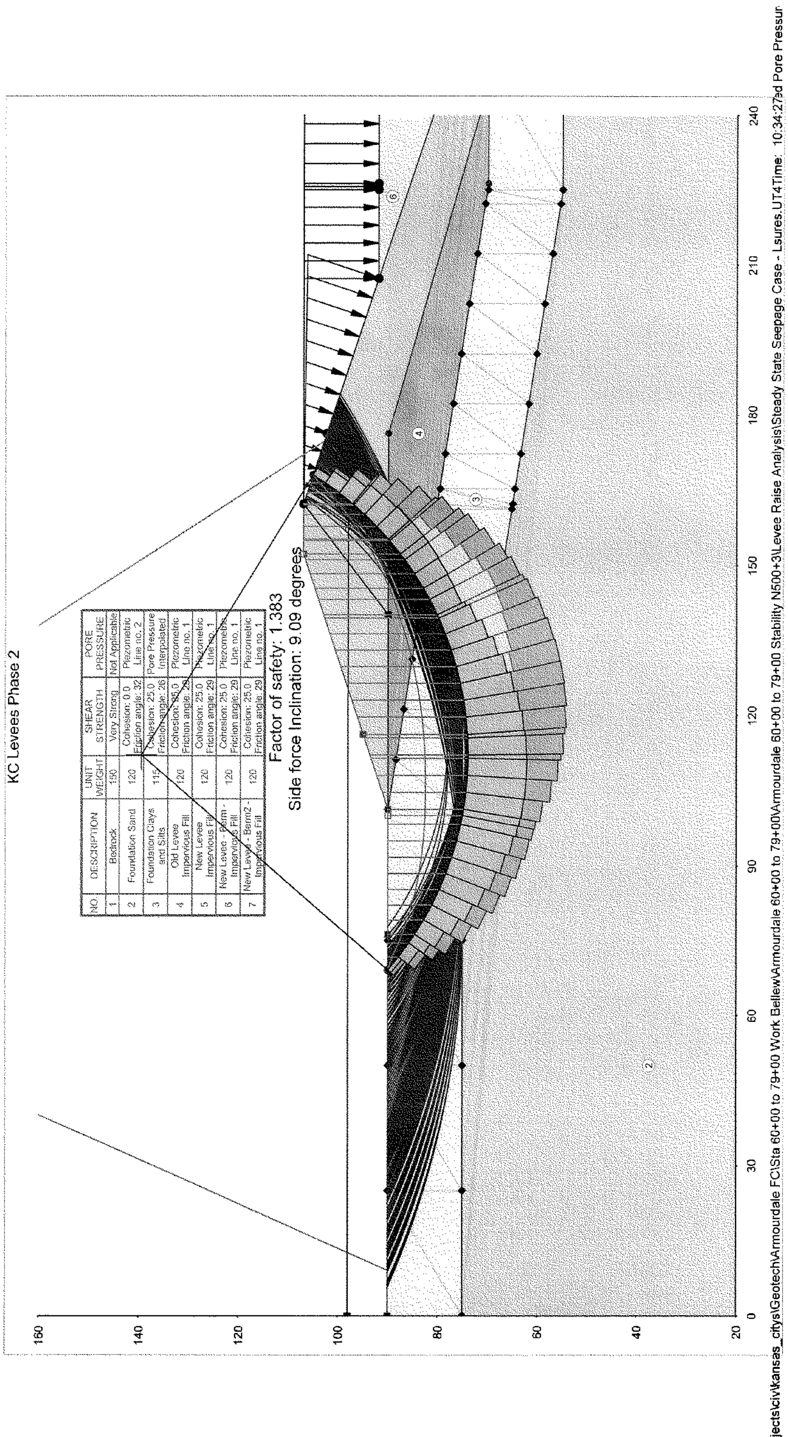

KC Levees Phase 2
NO. | DESCRIPTION | UNIT WEIGHT | SHEAR STRENGTH | PORE PRESSURE
1 | Bedrock | 150 | Very Strong | Not Applicable
2 | Foundation Sand | 120 | Cohesion: 0.0 Friction angle: 32 | Piezometric Line no. 2
3 | Foundation Clays and Silts | 115 | Cohesion: 25.0 Friction angle: 26 | Pore Pressure Interpolated
4 | Old Levee Impervious Fill | 120 | Cohesion: 25.0 Friction angle: 29 | Piezometric Line no. 1
5 | New Levee Impervious Fill | 120 | Cohesion: 25.0 Friction angle: 29 | Piezometric Line no. 1
6 | New Levee - Berm - Impervious Fill | 120 | Cohesion: 25.0 Friction angle: 29 | Piezometric Line no. 1
7 | New Levee - Berm2 - Impervious Fill | 120 | Cohesion: 25.0 Friction angle: 29 | Piezometric Line no. 1
Factor of safety: 1.383
Side force Inclination: 9.09 degrees
160
140
120
100
80
60
40
20
0
30
60
90
120
150
180
210
240
2
3
4
6
rojects\civ\kansas_citys\Geotech\Armourdale FC\Sta 60+00 to 79+00 Work Bellew\Armourdale 60+00 to 79+00 Stability N500+3\Levee Raise Analysis\Steady State Seepage Case - Lsures.UT4Time: 10:34:27sd Pore Pressur

29 MAR 2007
RSK

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HEADING
    Kansas City Levees Feasibility Study Phase II
    Armourdale Station 90+00
    End of Construction - n500 + 3 raise

PROFILE LINES
         1     1 New Levee Fill Moist
                 .00       76.50
               10.00       76.50
               76.00       60.00

         2     5 New Levee Fill Saturated
              -15.00       71.50
                 .00       76.50
               59.50       60.00

         3     2 Existing Levee Saturated
             -155.00       32.00
              -15.00       71.50
               -6.00       71.50
               28.50       60.00
               59.00       60.00

         4     3 Foundation Silt
             -300.00       25.00
             -180.00       25.00
             -155.00       32.00
             -145.00       32.00
              -82.00       50.00
                 .00       50.00
               59.00       60.00
              200.00       60.00

         5     4 Sand
             -300.00       10.00
             -180.00       10.00
              -50.00       35.00
                 .00       35.00
              200.00       35.00


MATERIAL PROPERTIES
     1 Levee Fill Moist
          115.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     2 Existing Levee Saturated
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     3 Foundation Silts
          115.00 Unit Weight
          Conventional Shear
              500.00        .00
          No Pore Pressure
     4 Foundation Sands
          120.00 Unit Weight
          Conventional Shear
                 .00      32.00
```

```
          Piezometric Line
          1
     5 New Levee Saturated
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure

PIEZOMETRIC LINES
         1      62.40 Groundwater Surface Elevation
              -300.00      25.00
              -180.00      25.00
               -92.00      50.00
                  .00      50.00
                59.50      60.00
               200.00      60.00
```

DISTRIBUTED LOADS
1
INTERPOLATION DATA
Pore Water Pressure

-300.00	25.00	3213.00	3
-300.00	15.00	3837.00	3
-300.00	10.00	4149.60	3
-222.00	25.00	3213.00	3
-222.00	15.00	3837.00	3
-222.00	10.00	4149.60	3
-180.00	25.00	3213.00	3
-180.00	10.00	4149.60	3
-180.00	15.00	3837.60	3
-155.00	32.00	2764.30	3
-155.00	21.00	[illegible]	3
-155.00	14.80	[illegible]	3
-82.00	[illegible]	[illegible]	3
-82.00	40.00	[illegible]	[illegible]
-82.00	28.80	2942.00	3
-50.00	[illegible]	[illegible]	3
-50.00	42.00	[illegible]	3
[illegible]	35.00	[illegible]	3
.00	50.00	1654.90	3
.00	[illegible]	2153.00	3
.00	[illegible]	2470.40	3
59.00	60.00	.00	3
[illegible]	48.00	749.00	3
[illegible]	35.00	2353.00	3
100.00	60.00	.00	3
100.00	48.00	749.00	3
100.00	35.00	2371.00	3
140.00	60.00	.00	3
140.00	48.00	749.00	3
140.00	35.00	2334.00	3
180.00	60.00	.00	3
180.00	48.00	749.00	3
180.00	35.00	2296.00	3
200.00	60.00	.00	3
200.00	48.00	749.00	3
200.00	35.00	2277.60	3

NOT USED FOR THIS ANALYSIS

ANALYSIS/COMPUTATION

```
     Circular Search 1
          60.00     100.00       1.00        .00
     Radius
          56.00
SINgle-stage Computations
ITEration limit
 50
TRIal max
 70
SAVe n most
 150
LONg-form output
CRAck
          12.00         D
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
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30 Mar 2007
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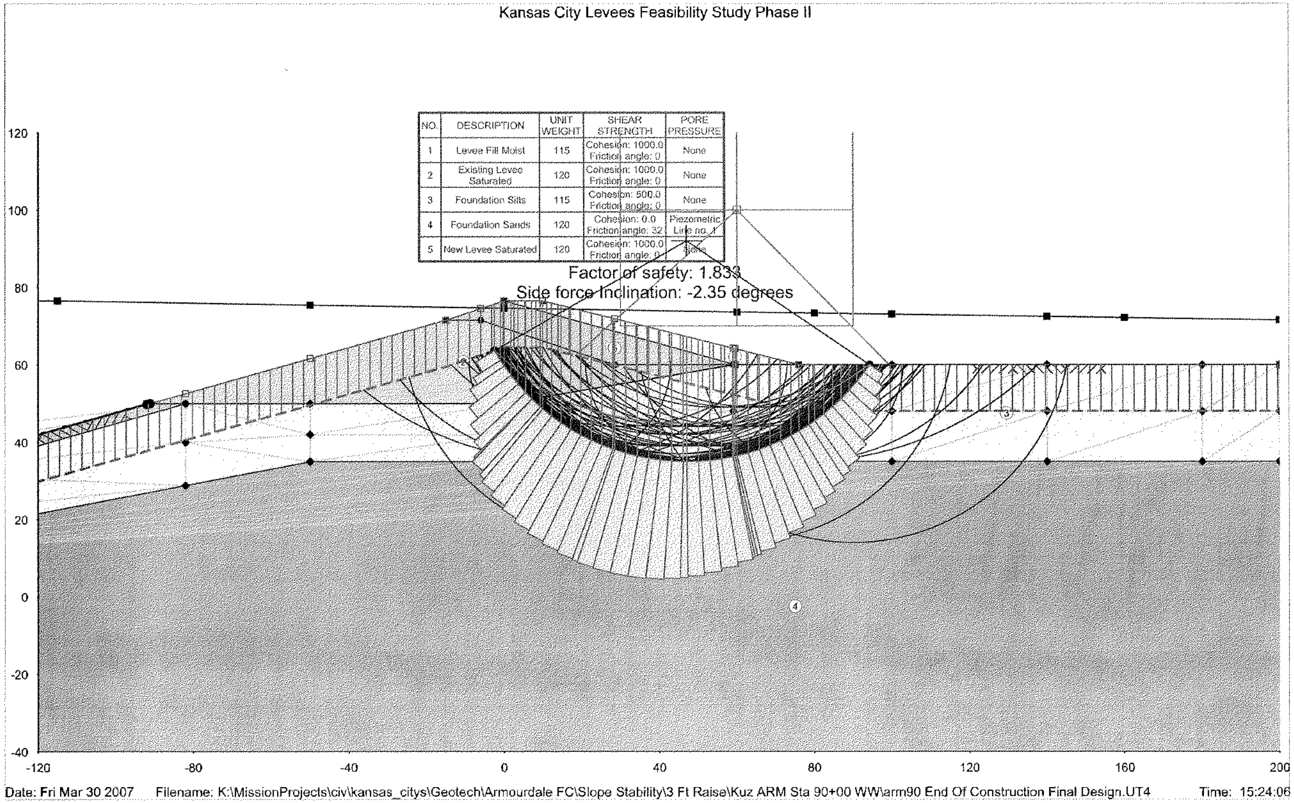

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Fill Moist	115	Cohesion: 1000.0 Friction angle: 0	None
2	Existing Levee Saturated	120	Cohesion: 1000.0 Friction angle: 0	None
3	Foundation Silts	115	Cohesion: 500.0 Friction angle: 0	None
4	Foundation Sands	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
5	New Levee Saturated	120	Cohesion: 1000.0 Friction angle: 0	None

29 MAR 2007
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HEADING
    Kansas City Levees Feasibility Study Phase II
    Armourdale Station 90+00
    Steady state Seepage - n500 + 3 raise

PROFILE LINES
         1     1 New Levee Fill Moist
                 .00       76.50
               10.00       76.50
               76.00       60.00

         2     5 New Levee Fill Saturated
              -15.00       71.50
                 .00       76.50
               59.50       60.00

         3     2 Existing Levee Saturated
             -155.00       32.00
              -15.00       71.50
               -6.00       71.50
               28.50       60.00
               59.00       60.00

         4     3 Foundation Silt
             -300.00       25.00
             -180.00       25.00
             -155.00       32.00
             -145.00       32.00
              -82.00       50.00
                 .00       50.00
               59.00       60.00
              200.00       60.00

         5     4 Sand
             -300.00       10.00
             -180.00       10.00
              -50.00       35.00
                 .00       35.00
              200.00       35.00


MATERIAL PROPERTIES
      1 Levee Fill Moist
           115.00 Unit Weight
           Conventional Shear
                 .00       29.00
           No Pore Pressure
      2 Existing Levee Saturated
           120.00 Unit Weight
           Conventional Shear
                 .00       29.00
           Piezometric Line
           1
      3 Foundation Silts
           115.00 Unit Weight
           Conventional Shear
                 .00       26.00
           Interpolate Pore Water Pressure
      4 Foundation Sands
           120.00 Unit Weight
           Conventional Shear
```
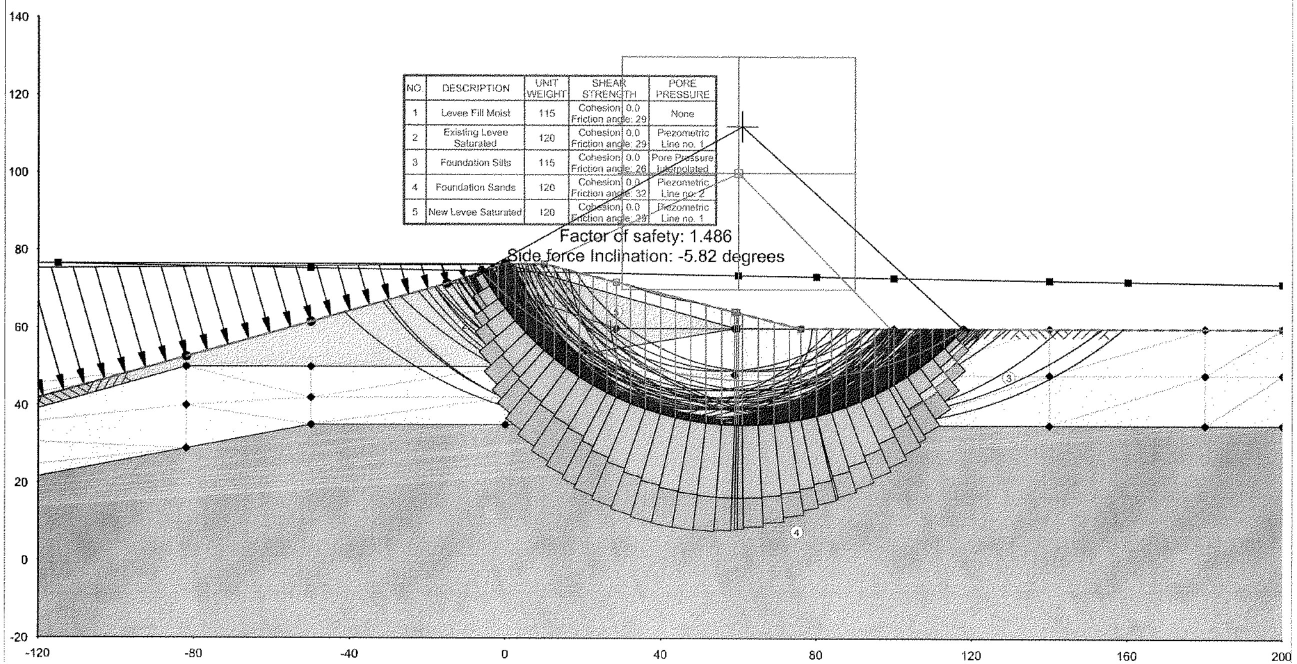

```
               .00      32.00
          Piezometric Line
          2
     5 New Levee Saturated
          120.00 Unit Weight
          Conventional Shear
               .00      29.00
          Piezometric Line
          1

PIEZOMETRIC LINES
         1      62.40 Water Surface Elev.
             -300.00        76.50
                 .00        76.50
               59.50        60.00
              200.00        60.00

         2      62.40 Foundation Sand Hydraulic Grade Line
             -300.00        76.50
             -115.00        76.50
              -50.00        75.40
                 .00        74.60
               60.00        73.60
               80.00        73.30
              100.00        73.00
              140.00        72.40
              160.00        72.10
              200.00        71.50


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
          -300.00       25.00    3213.00          3
          -300.00       15.00    3837.00          3
          -300.00       10.00    4149.60          3
          -222.00       25.00    3213.00          3
          -222.00       15.00    3837.00          3
          -222.00       10.00    4149.60          3
          -180.00       25.00    3213.00          3
          -180.00       10.00    4149.60          3
          -180.00       15.00    3837.60          3
          -155.00       32.00    2764.30          3
          -155.00       21.00    3463.00          3
          -155.00       14.80    3850.00          3
           -82.00       50.00    1653.00          3
           -82.00       40.00    2777.00          3
           -82.00       28.80    2942.00          3
           -50.00       50.00    1654.00          3
           -50.00       42.00    2153.00          3
           -50.00       35.00    2556.00          3
              .00       50.00    1654.90          3
              .00       42.00    2153.00          3
              .00       35.00    2470.40          3
            59.00       60.00        .00          3
            59.00       48.00     749.00          3
            59.00       35.00    2353.00          3
           100.00       60.00        .00          3
           100.00       48.00     749.00          3
           100.00       35.00    2371.00          3
```

```
        140.00      60.00        .00          3
        140.00      48.00     749.00          3
        140.00      35.00    2334.00          3
        180.00      60.00        .00          3
        180.00      48.00     749.00          3
        180.00      35.00    2296.00          3
        200.00      60.00        .00          3
        200.00      48.00     749.00          3
        200.00      35.00    2277.60          3


ANALYSIS/COMPUTATION
     Circular Search 1
          60.00    100.00       1.00        .00
     Radius
          56.00
SINgle-stage Computations
ITEration limit
 50
TRIal max
 70
SAVe n most
 150
LONg-form output
CRAck
           .00         D
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

29 Mar 2007 (4)

Kansas City Levees Feasibility Study Phase II



Date: Thu Mar 29 2007 Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 90+00 WW\arm90 Steady Seepage Final Design.UT4 Time: 15:38:41

31 MAR 2007
RSK

```
HEADING
    Armourdale Station 245+50 n500+3
    Station 245+50
    End of Construction conditions

PROFILE LINES
         1     1 New Levee Fill Moist
                  .00        68.10
                10.00        68.10
                90.00        48.00

         2     5 New Levee Fill Saturated
               -10.00        64.30
                  .00        68.10
                65.00        48.00

         3     2 Existing Levee Saturated
               -94.00        38.00
               -33.00        55.00
               -10.00        64.30
                  .00        64.30
                35.00        50.00

         4     3 Foundation Silt
              -172.00        30.00
              -144.00        38.00
               -94.00        38.00
                35.00        50.00
                55.00        50.00
                65.00        48.00
               200.00        48.00

         5     4 Sand
              -300.00        20.00
              -207.00        20.00
              -172.00        30.00
               200.00        30.00

         6     1 Berm 1
                58.16        56.00
                78.00        56.00
               110.00        48.00

         7     1 Berm 2
                94.00        52.00
               104.00        52.00
               118.00        48.00


MATERIAL PROPERTIES
     1 Levee Fill Moist
          115.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     2 Existing Levee Saturated
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     3 Foundation Silts
```

```
        115.00 Unit Weight
        Conventional Shear
            500.00        .00
        No Pore Pressure
   4 Foundation Sands
        120.00 Unit Weight
        Conventional Shear
              .00     32.00
        Piezometric Line
        1
   5 New Levee Saturated
        120.00 Unit Weight
        Conventional Shear
           1000.00        .00
        No Pore Pressure
```

PIEZOMETRIC LINES

```
        1      62.40 Water Surface Elev.
           -207.00      20.00
           -172.00      30.00
           -144.00      38.00
            -94.00      38.00
             35.00      50.00
             55.00      50.00
             65.00      48.00
            200.00      48.00
```

INTERPOLATION DATA

Pore Water Pressure

-172.00	30.00	2377.40	3
-144.00	38.00	1878.20	3
-144.00	30.00	2377.40	3
-120.00	38.00	1878.20	3
-120.00	30.00	234[illegible].00	3
-94.00	38.00	18[illegible]8.2[illegible]	3
-94.00	30.00	228[illegible].[illegible]0	3
-64.00	40.7[illegible]	1646.80	3
-64.00	[illegible]	2209.00	3
-33.0[illegible]	[illegible]7	1405.[illegible]	3
-33.00	[illegible]0.00	2[illegible]1[illegible]0	3
.00	46.74	[illegible]33.50	3
.00	30.[illegible]	2021.80	3
35.00	[illegible]	512.90	3
35.00	[illegible].00	1921.90	3
65.00	48.00	.00	3
[illegible]	30.00	1872.00	3
[illegible].00	48.00	.00	3
90.00	30.00	1847.00	3
135.00	30.00	1809.60	3
135.00	48.00	.00	3
185.00	48.00	.00	3
185.00	30.00	1778.40	3
235.00	48.00	.00	3
235.00	30.00	1747.20	3
285.00	48.00	.00	3
285.00	30.00	1722.20	3

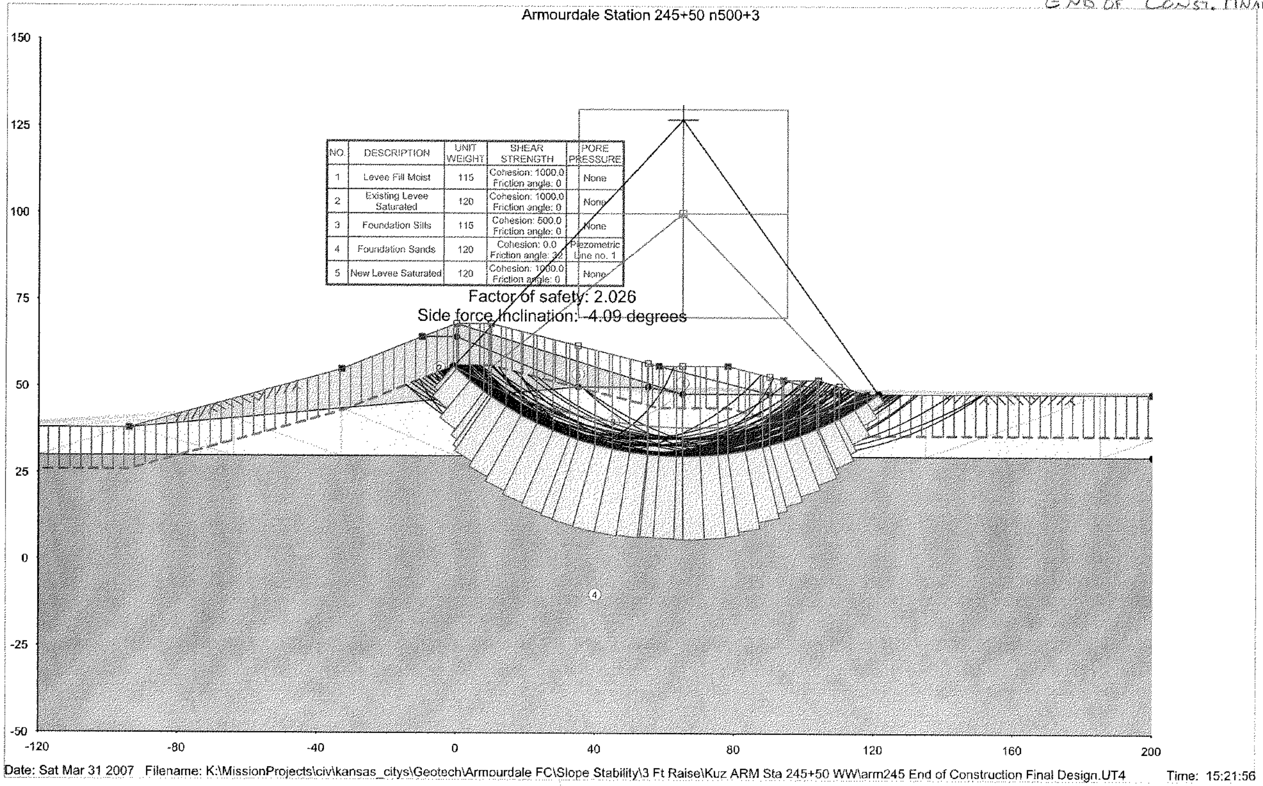
NOT USED FOR THIS ANALYSIS

ANALYSIS/COMPUTATION

```
     Circular Search 1
```

```
          65.00     100.00        1.00         .00
     Radius
          70.00
SINgle-stage Computations
ITEration limit
 50
SAVe n most
 100
LONg-form output
CRAck
          12.00         D
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

31 MAR 2007 ①
END OF CONST. FINAL DESIGN



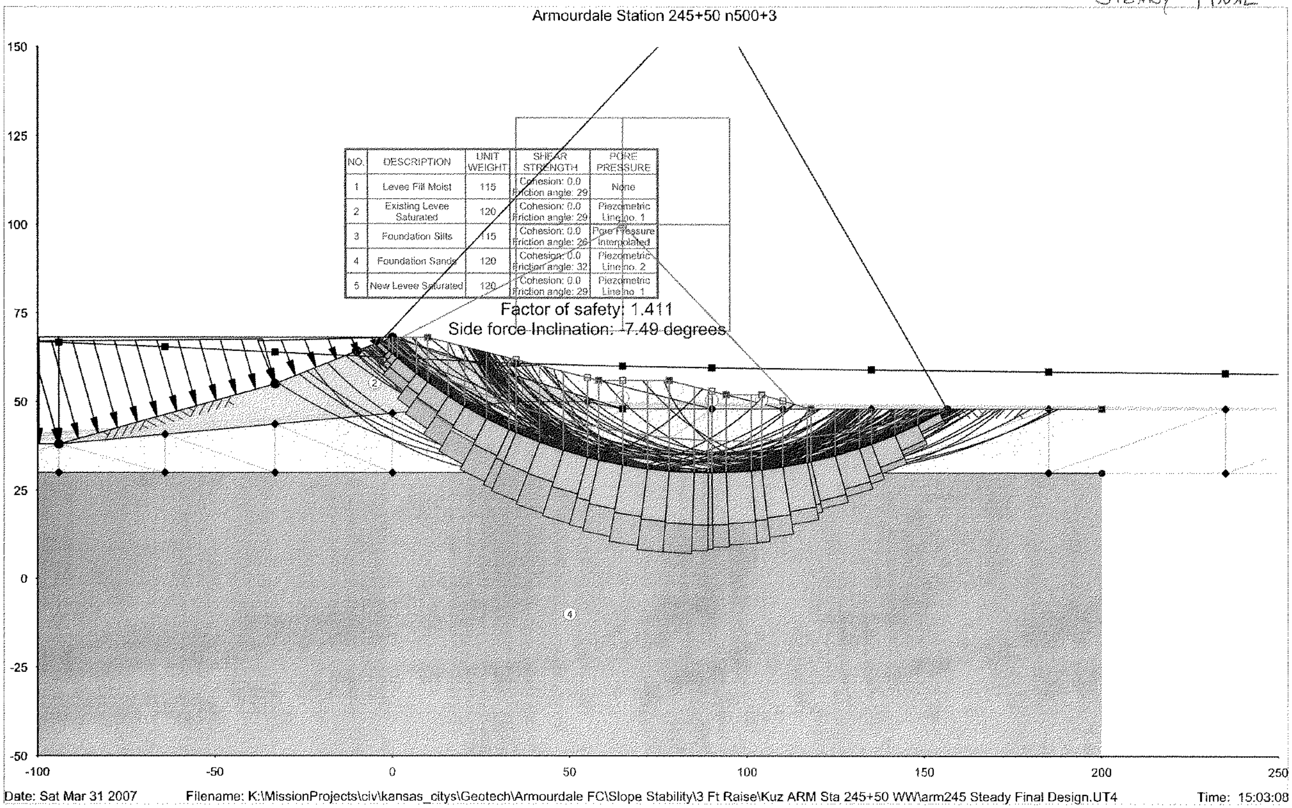
31 March 2007
RSK

```
HEADING
    Armourdale Station 245+50 n500+3
    Station 245+50
    Steady state Seepage conditions

PROFILE LINES
        1     1 New Levee Fill Moist
                .00       68.10
              10.00       68.10
              90.00       48.00

        2     5 New Levee Fill Saturated
             -10.00       64.30
                .00       68.10
              65.00       48.00

        3     2 Existing Levee Saturated
             -94.00       38.00
             -33.00       55.00
             -10.00       64.30
                .00       64.30
              35.00       50.00

        4     3 Foundation Silt
            -172.00       30.00
            -144.00       38.00
             -94.00       38.00
              35.00       50.00
              55.00       50.00
              65.00       48.00
             200.00       48.00

        5     4 Sand
            -300.00       20.00
            -207.00       20.00
            -172.00       30.00
             200.00       30.00

        6     1 Berm 1
              58.16       56.00
              78.00       56.00
             110.00       48.00

        7     1 Berm 2
              94.00       52.00
             104.00       52.00
             118.00       48.00

MATERIAL PROPERTIES
     1 Levee Fill Moist
          115.00 Unit Weight
          Conventional Shear
                 .00      29.00
          No Pore Pressure
     2 Existing Levee Saturated
          120.00 Unit Weight
          Conventional Shear
                 .00      29.00
          Piezometric Line
          1
```

```
      3 Foundation Silts
           115.00 Unit Weight
           Conventional Shear
                  .00     26.00
           Interpolate Pore Water Pressure
      4 Foundation Sands
           120.00 Unit Weight
           Conventional Shear
                  .00     32.00
           Piezometric Line
           2
      5 New Levee Saturated
           120.00 Unit Weight
           Conventional Shear
                  .00     29.00
           Piezometric Line
           1

PIEZOMETRIC LINES
        1      62.40 Water Surface Elev.
            -300.00      68.10
                .00      68.10
              65.00      48.00
             200.00      48.00

        2      62.40 Foundation Sand Hydraulic Grade Line
            -300.00      68.10
            -144.00      68.10
            -120.00      67.50
             -94.00      66.60
             -64.00      65.40
             -33.00      63.90
                .00      62.40
              35.00      60.80
              65.00      60.00
              90.00      59.60
             135.00      59.00
             185.00      58.50
             235.00      58.00
             285.00      57.60


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
        -172.00     30.00   2377.40         3
        -144.00     38.00   1878.20         3
        -144.00     30.00   2377.40         3
        -120.00     38.00   1878.20         3
        -120.00     30.00   2340.00         3
         -94.00     38.00   1878.20         3
         -94.00     30.00   2283.80         3
         -64.00     40.79   1646.80         3
         -64.00     30.00   2209.00         3
         -33.00     43.67   1405.70         3
         -33.00     30.00   2115.40         3
            .00     46.74   1133.50         3
            .00     30:00   2021.80         3
          35.00     50.00    512.90         3
          35.00     30.00   1921.90         3
```

```
          65.00      48.00        .00          3
          65.00      30.00    1872.00          3
          90.00      48.00        .00          3
          90.00      30.00    1847.00          3
         135.00      30.00    1809.60          3
         135.00      48.00        .00          3
         185.00      48.00        .00          3
         185.00      30.00    1778.40          3
         235.00      48.00        .00          3
         235.00      30.00    1747.20          3
         285.00      48.00        .00          3
         285.00      30.00    1722.20          3


ANALYSIS/COMPUTATION
     Circular Search 1
          65.00     100.00       1.00        .00
     Radius
          70.00
SINgle-stage Computations
ITEration limit
 50
SAVe n most
 100
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

31 MARCH 2007
STEADY FINAL
(6)

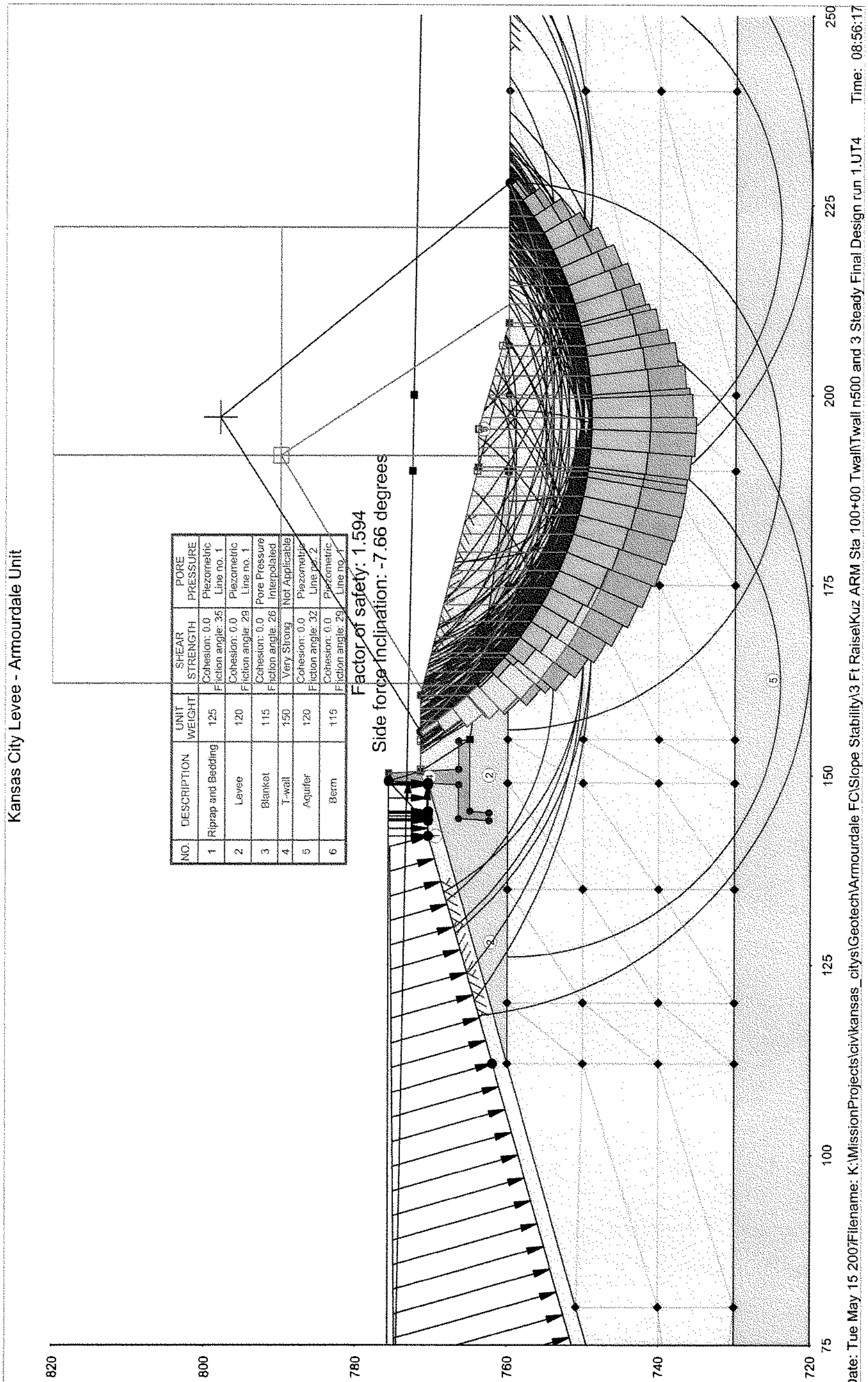


FINAL DESIGN RUN 1.DAT

```
HEADING
    Kansas City Levee - Armourdale Unit
    Feasibility Study
    Phase II
    Global Stability for T-wall on Levee Raise
    N500 + 3 Design Condition - STA 100+00

PROFILE LINES
        1     1 Riprap and Bedding
                 .00     730.00
              142.00     770.50
              149.00     770.50

        2     2 Levee 1
              112.00     760.00
              149.00     770.50

        3     6 Levee 2
              150.50     771.50
              160.50     771.50
              206.50     760.00

        4     2 Levee 3
              144.00     762.50
              145.00     762.50
              145.25     765.00
              154.75     765.00
              190.00     760.00

        5     3 Blanket
                7.00     730.00
              112.00     760.00
              300.00     760.00

        6     4 T-wall
              144.00     762.50
              144.25     766.50
              148.75     766.50
              149.00     770.50
              149.25     775.70
              150.25     775.70
              150.50     771.50
              150.75     766.50
              154.50     766.50
              154.75     765.00

        8     6 Berm
              190.50     764.00
              195.50     764.00
              209.50     760.00

        7     5 Aquifer
                 .00     730.00
              300.00     730.00


MATERIAL PROPERTIES
     1 Riprap and Bedding
          125.00 Unit Weight
          Conventional Shear
                 .00      35.00
```

```
          Piezometric Line
          1
     2 Levee
          120.00 Unit Weight
          Conventional Shear
                 .00      29.00
          Piezometric Line
          1
     3 Blanket
          115.00 Unit Weight
          Conventional Shear
                 .00      26.00
          Interpolate Pore Water Pressure
     4 T-wall
          150.00 Unit Weight
          Very Strong
     5 Aquifer
          120.00 Unit Weight
          Conventional Shear
                 .00      32.00
          Piezometric Line
          2
     6 Berm
          115.00 Unit Weight
          Conventional Shear
                 .00      29.00
          Piezometric Line
          1

PIEZOMETRIC LINES
         1      62.40 Piezometric Line 1
                  .00     775.70
               149.25     775.70
               150.50     771.50
               154.75     765.00
               190.00     760.00
               300.00     760.00

         2      62.40 Piezometric Line 2
                  .00     775.70
                 7.00     775.70
               190.00     772.65
               200.00     772.50
               300.00     771.00


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
            7.00     730.00     2851.70          3
           40.00     739.43     2263.25          3
           40.00     735.00     2520.60          3
           40.00     730.00     2811.10          3
           80.00     750.86     1550.00          3
           80.00     740.00     2185.50          3
           80.00     730.00     2770.60          3
          112.00     760.00      979.70          3
          112.00     750.00     1566.30          3
          112.00     740.00     2152.80          3
          112.00     730.00     2739.40          3
```

```
         120.00      760.00      979.70           3
         120.00      750.00     1563.10           3
         120.00      740.00     2146.60           3
         120.00      730.00     2730.00           3
         135.00      760.00      979.70           3
         135.00      750.00     1557.90           3
         135.00      740.00     2136.20           3
         135.00      730.00     2714.40           3
         149.00      760.00      979.70           3
         149.00      750.00     1553.80           3
         149.00      740.00     2127.80           3
         149.00      730.00     2701.90           3
         154.75      760.00      385.10           3
         154.75      750.00     1155.30           3
         154.75      740.00     1925.50           3
         154.75      730.00     2695.70           3
         175.00      760.00      175.10           3
         175.00      750.00     1009.10           3
         175.00      730.00     2677.00           3
         175.00      740.00     1843.00           3
         190.00      760.00         .00           3
         190.00      750.00      887.10           3
         190.00      740.00     1774.30           3
         190.00      730.00     2661.40           3
         200.00      760.00         .00           3
         200.00      750.00      884.00           3
         200.00      740.00     1768.00           3
         200.00      730.00     2652.00           3
         240.00      760.00         .00           3
         240.00      750.00      871.50           3
         240.00      730.00     2614.60           3
         240.00      740.00     1743.10           3
         280.00      760.00         .00           3
         280.00      750.00      859.00           3
         280.00      740.00     1718.00           3
         280.00      730.00     2577.10           3
         300.00      760.00         .00           3
         300.00      750.00      852.80           3
         300.00      740.00     1705.60           3
         300.00      730.00     2558.40           3

ANALYSIS/COMPUTATION
     Circular Search 1
         192.00      790.00        1.00      670.00
     Radius
          36.00
SINgle-stage Computations
SAVe n most
 150
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```



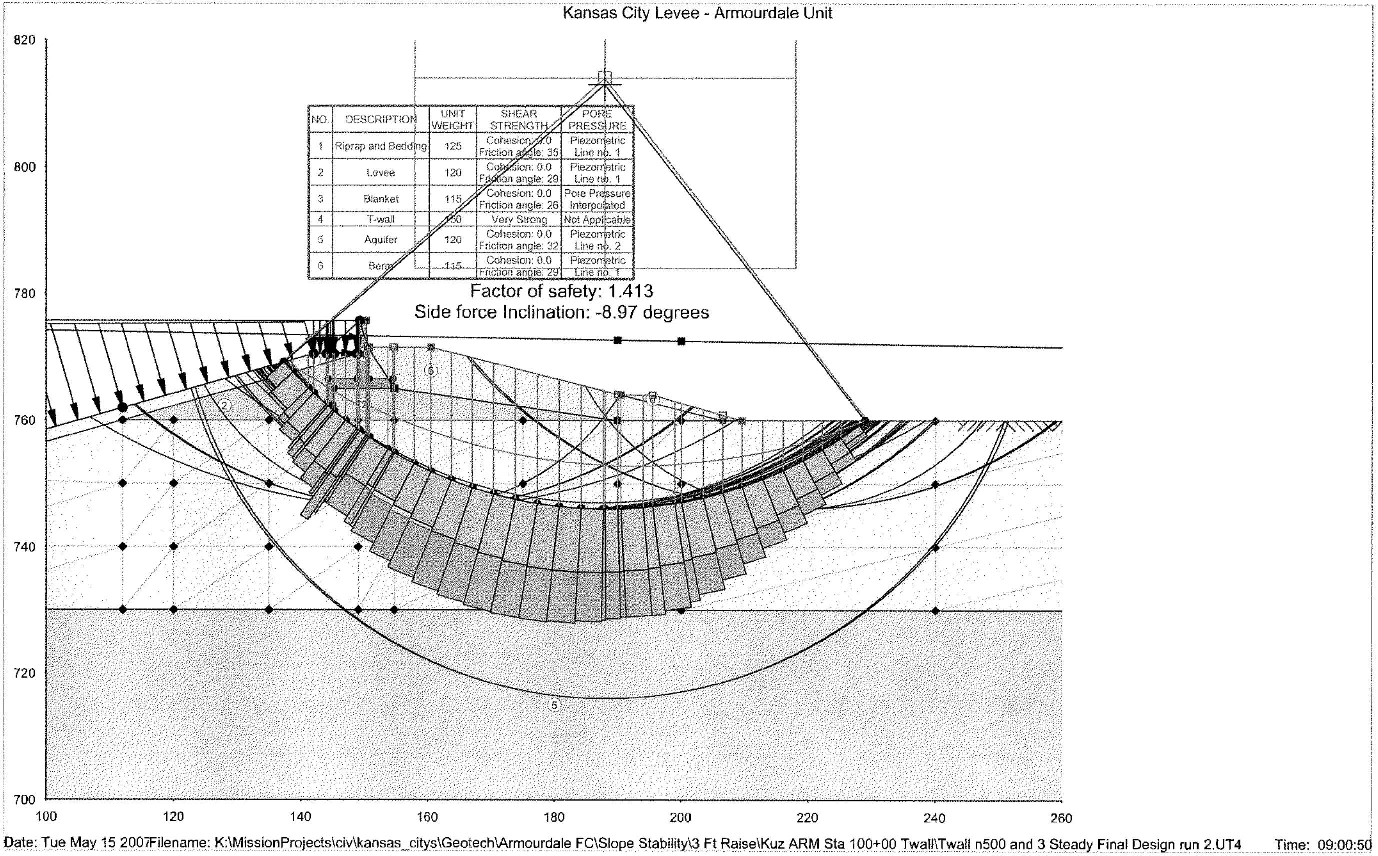
Kansas City Levee - Armourdale Unit
NO. | DESCRIPTION | UNIT WEIGHT | SHEAR STRENGTH | PORE PRESSURE
1 | Riprap and Bedding | 125 | Cohesion: 0.0 Friction angle: 35 | Piezometric Line no. 1
2 | Levee | 120 | Cohesion: 0.0 Friction angle: 29 | Piezometric Line no. 1
3 | Blanket | 115 | Cohesion: 0.0 Friction angle: 26 | Pore Pressure Interpolated
4 | T-wall | 150 | Very Strong | Not Applicable
5 | Aquifer | 120 | Cohesion: 0.0 Friction angle: 32 | Piezometric Line no. 2
6 | Berm | 115 | Cohesion: 0.0 Friction angle: 29 | Piezometric Line no. 1
Factor of safety: 1.594
Side force inclination: -7.66 degrees
820
800
780
760
740
720
75
100
125
150
175
200
225
250
Date: Tue May 15 2007
Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 100+00 Twall\Twall n500 and 3 Steady Final Design run 1.UT4
Time: 08:56:17

FINAL DESIGN RUN 2 DR

```
HEADING
    Kansas City Levee - Armourdale Unit
    Feasibility Study
    Phase II
    Global Stability for T-wall on Levee Raise
    N500 + 3 Design Condition  STA 100+00

PROFILE LINES
        1      1 Riprap and Bedding
                  .00     730.00
               142.00     770.50
               149.00     770.50

        2      2 Levee 1
               112.00     760.00
               149.00     770.50

        3      6 Levee 2
               150.50     771.50
               160.50     771.50
               206.50     760.00

        4      2 Levee 3
               144.00     762.50
               145.00     762.50
               145.25     765.00
               154.75     765.00
               190.00     760.00

        5      3 Blanket
                 7.00     730.00
               112.00     760.00
               300.00     760.00

        6      4 T-wall
               144.00     762.50
               144.25     766.50
               148.75     766.50
               149.00     770.50
               149.25     775.70
               150.25     775.70
               150.50     771.50
               150.75     766.50
               154.50     766.50
               154.75     765.00

        8      6 Berm
               190.50     764.00
               195.50     764.00
               209.50     760.00

        7      5 Aquifer
                  .00     730.00
               300.00     730.00


MATERIAL PROPERTIES
     1 Riprap and Bedding
          125.00 Unit Weight
          Conventional Shear
                 .00      35.00
```

```
          Piezometric Line
          1
     2 Levee
          120.00 Unit Weight
          Conventional Shear
                 .00      29.00
          Piezometric Line
          1
     3 Blanket
          115.00 Unit Weight
          Conventional Shear
                 .00      26.00
          Interpolate Pore Water Pressure
     4 T-wall
          150.00 Unit Weight
          Very Strong
     5 Aquifer
          120.00 Unit Weight
          Conventional Shear
                 .00      32.00
          Piezometric Line
          2
     6 Berm
          115.00 Unit Weight
          Conventional Shear
                 .00      29.00
          Piezometric Line
          1

PIEZOMETRIC LINES
         1      62.40 Piezometric Line 1
                  .00     775.70
               149.25     775.70
               150.50     771.50
               154.75     765.00
               190.00     760.00
               300.00     760.00

         2      62.40 Piezometric Line 2
                  .00     775.70
                 7.00     775.70
               190.00     772.65
               200.00     772.50
               300.00     771.00


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
           7.00     730.00    2851.70          3
          40.00     739.43    2263.25          3
          40.00     735.00    2520.60          3
          40.00     730.00    2811.10          3
          80.00     750.86    1550.00          3
          80.00     740.00    2185.50          3
          80.00     730.00    2770.60          3
         112.00     760.00     979.70          3
         112.00     750.00    1566.30          3
         112.00     740.00    2152.80          3
         112.00     730.00    2739.40          3
```

```
        120.00     760.00     979.70           3
        120.00     750.00    1563.10           3
        120.00     740.00    2146.60           3
        120.00     730.00    2730.00           3
        135.00     760.00     979.70           3
        135.00     750.00    1557.90           3
        135.00     740.00    2136.20           3
        135.00     730.00    2714.40           3
        149.00     760.00     979.70           3
        149.00     750.00    1553.80           3
        149.00     740.00    2127.80           3
        149.00     730.00    2701.90           3
        154.75     760.00     385.10           3
        154.75     750.00    1155.30           3
        154.75     740.00    1925.50           3
        154.75     730.00    2695.70           3
        175.00     760.00     175.10           3
        175.00     750.00    1009.10           3
        175.00     730.00    2677.00           3
        175.00     740.00    1843.00           3
        190.00     760.00        .00           3
        190.00     750.00     887.10           3
        190.00     740.00    1774.30           3
        190.00     730.00    2661.40           3
        200.00     760.00        .00           3
        200.00     750.00     884.00           3
        200.00     740.00    1768.00           3
        200.00     730.00    2652.00           3
        240.00     760.00        .00           3
        240.00     750.00     871.50           3
        240.00     730.00    2614.60           3
        240.00     740.00    1743.10           3
        280.00     760.00        .00           3
        280.00     750.00     859.00           3
        280.00     740.00    1718.00           3
        280.00     730.00    2577.10           3
        300.00     760.00        .00           3
        300.00     750.00     852.80           3
        300.00     740.00    1705.60           3
        300.00     730.00    2558.40           3

ANALYSIS/COMPUTATION
     Circular Search 1
        188.00     814.00       1.00      670.00
     Radius
         68.00
SINGle-stage Computations
SAVe n most
 150
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```



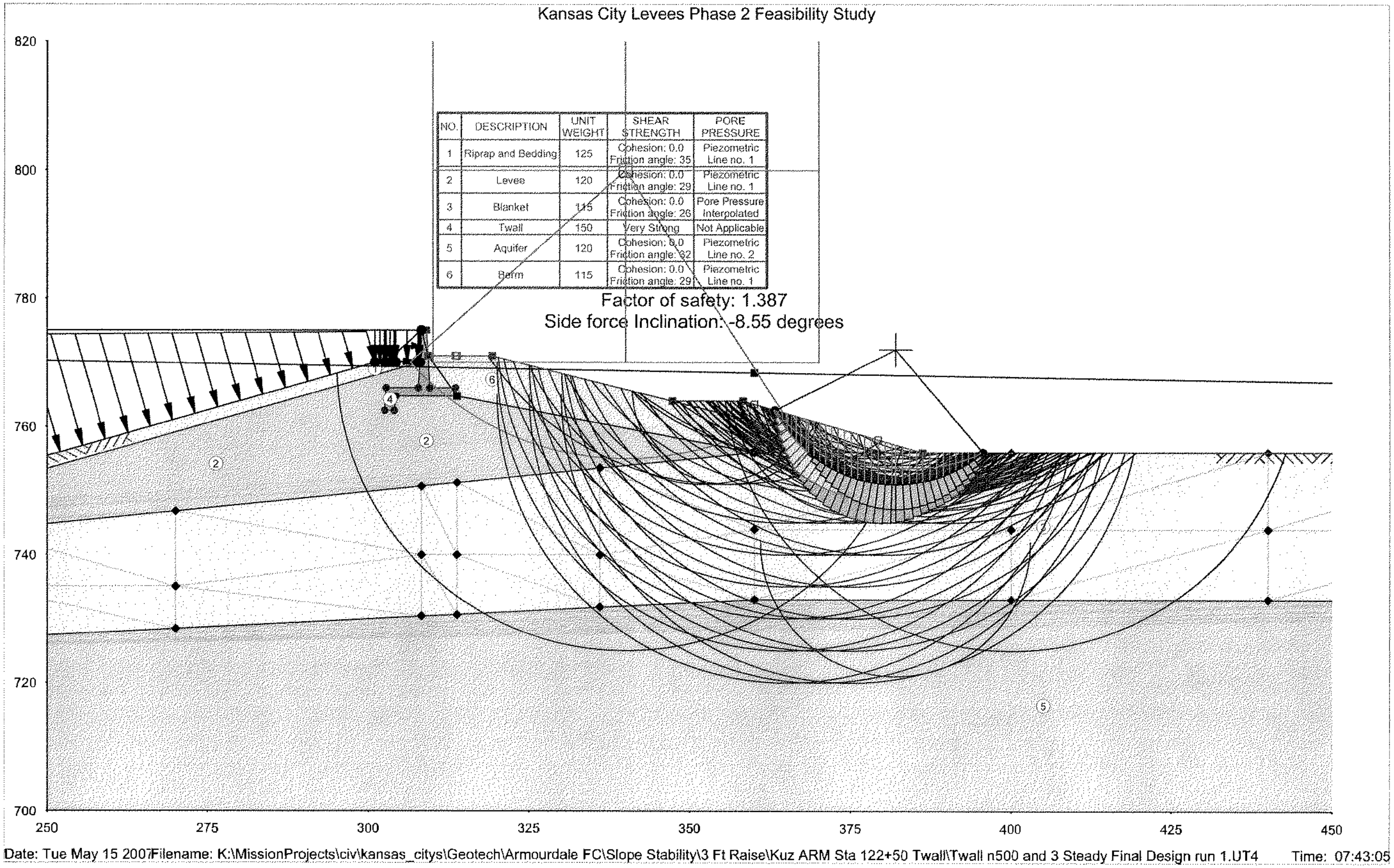
Kansas City Levee - Armourdale Unit
NO. DESCRIPTION UNIT WEIGHT SHEAR STRENGTH PORE PRESSURE
1 Riprap and Bedding 125 Cohesion: 0.0 Friction angle: 35 Piezometric Line no. 1
2 Levee 120 Cohesion: 0.0 Friction angle: 29 Piezometric Line no. 1
3 Blanket 115 Cohesion: 0.0 Friction angle: 26 Pore Pressure Interpolated
4 T-wall 150 Very Strong Not Applicable
5 Aquifer 120 Cohesion: 0.0 Friction angle: 32 Piezometric Line no. 2
6 Berm 115 Cohesion: 0.0 Friction angle: 29 Piezometric Line no. 1
Factor of safety: 1.413
Side force Inclination: -8.97 degrees
820
800
780
760
740
720
700
100
120
140
160
180
200
220
240
260
Date: Tue May 15 2007Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 100+00 Twall\Twall n500 and 3 Steady Final Design run 2.UT4
Time: 09:00:50

FINAL DESIGN RUN 1.DAT

```
HEADING
    Kansas City Levees Phase 2 Feasibility Study
    Armourdale Unit
    T-wall on Levee Station 122+50
    Global Stability Steady Seepage Condition

PROFILE LINES
        8     6 Berm
              347.25     764.00
              358.25     764.00
              386.25     756.00

        2     2 Levee 1
              203.00     740.00
              308.00     770.00

        3     6 Levee 2
              309.25     771.00
              319.25     771.00
              379.25     756.00

        4     2 Levee 3
              302.50     762.50
              304.00     762.50
              304.25     764.75
              313.75     764.75
              360.00     756.00

        5     3 Blanket
              100.00     740.00
              203.00     740.00
              360.00     756.00
              500.00     756.00

        6     4 Twall
              302.50     762.50
              302.75     766.00
              307.75     766.00
              308.00     770.00
              308.25     775.00
              309.00     775.00
              309.25     771.00
              309.50     766.00
              313.50     766.00
              313.75     764.75

        7     5 Aquifer
              100.00     725.00
              203.00     725.00
              360.00     733.00
              500.00     733.00

        1     1 Riprap and Bedding
              196.00     740.00
              301.00     770.00
              308.00     770.00


MATERIAL PROPERTIES
     1 Riprap and Bedding
          125.00 Unit Weight
```

```
          Conventional Shear
                 .00      35.00
          Piezometric Line
          1
     2 Levee
          120.00 Unit Weight
          Conventional Shear
                 .00      29.00
          Piezometric Line
          1
     3 Blanket
          115.00 Unit Weight
          Conventional Shear
                 .00      26.00
          Interpolate Pore Water Pressure
     4 Twall
          150.00 Unit Weight
          Very Strong
     5 Aquifer
          120.00 Unit Weight
          Conventional Shear
                 .00      32.00
          Piezometric Line
          2
     6 Berm
          115.00 Unit Weight
          Conventional Shear
                 .00      29.00
          Piezometric Line
          1

PIEZOMETRIC LINES
         1      62.40 Piezometric Line 1
              100.00     775.00
              308.25     775.00
              309.25     771.00
              313.75     764.75
              360.00     756.00
              600.00     756.00

         2      62.40 Piezometric Line 2
               85.00     773.00
              360.00     768.40
              460.00     766.80
              560.00     765.40
              600.00     764.70


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
         120.00     740.00    2184.00          3
         120.00     733.00    2545.10          3
         120.00     725.00    2957.80          3
         160.00     740.00    2184.00          3
         160.00     733.00    2524.70          3
         160.00     725.00    2914.00          3
         203.00     740.00    2184.00          3
         203.00     733.00    2650.80          3
         203.00     725.00    2870.40          3
```

```
         240.00     743.80    1864.80          3
         240.00     735.00    2307.20          3
         240.00     726.90    2714.40          3
         270.00     746.80    1634.40          3
         270.00     735.00    2251.00          3
         270.00     728.40    2595.80          3
         308.25     750.70    1516.30          3
         308.25     740.00    1996.50          3
         308.25     730.40    2427.40          3
         313.75     751.30     839.30          3
         313.75     740.00    1696.00          3
         313.75     730.60    2408.60          3
         336.00     753.60     433.70          3
         336.00     740.00    1603.50          3
         336.00     731.80    2308.80          3
         360.00     756.00        .00          3
         360.00     744.00    1152.50          3
         360.00     733.00    2209.00          3
         400.00     756.00        .00          3
         400.00     744.00    1038.50          3
         400.00     733.00    2171.50          3
         440.00     756.00        .00          3
         440.00     744.00    1110.20          3
         440.00     733.00    2127.80          3
         480.00     756.00        .00          3
         480.00     744.00    1090.60          3
         480.00     733.00    2090.40          3
         520.00     756.00        .00          3
         520.00     744.00    1074.40          3
         520.00     733.00    2059.20          3
         560.00     756.00        .00          3
         560.00     744.00    1054.90          3
         560.00     733.00    2021.80          3
         600.00     756.00        .00          3
         600.00     744.00    1032.10          3
         600.00     733.00    1978.10          3

ANALYSIS/COMPUTATION
     Circular Search 1
         340.00     800.00       1.00     670.00
     Radius
         45.00
SINgle-stage Computations
SAVe n most
 150
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```



Kansas City Levees Phase 2 Feasibility Study
NO. | DESCRIPTION | UNIT WEIGHT | SHEAR STRENGTH | PORE PRESSURE
1 | Riprap and Bedding | 125 | Cohesion: 0.0 Friction angle: 35 | Piezometric Line no. 1
2 | Levee | 120 | Cohesion: 0.0 Friction angle: 29 | Piezometric Line no. 1
3 | Blanket | 115 | Cohesion: 0.0 Friction angle: 26 | Pore Pressure Interpolated
4 | Twall | 150 | Very Strong | Not Applicable
5 | Aquifer | 120 | Cohesion: 0.0 Friction angle: 32 | Piezometric Line no. 2
6 | Berm | 115 | Cohesion: 0.0 Friction angle: 29 | Piezometric Line no. 1
Factor of safety: 1.387
Side force Inclination: -8.55 degrees
820
800
780
760
740
720
700
250
275
300
325
350
375
400
425
450
Date: Tue May 15 2007
Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 122+50 Twall\Twall n500 and 3 Steady Final Design run 1.UT4
Time: 07:43:05

Final Design Run 2.DAT

```
HEADING
    Kansas City Levees Phase 2 Feasibility Study
    Armourdale Unit
    T-wall on Levee Station 122+50
    Global Stability Steady Seepage Condition

PROFILE LINES
         8    6 Berm
              347.25     764.00
              358.25     764.00
              386.25     756.00

         2    2 Levee 1
              203.00     740.00
              308.00     770.00

         3    6 Levee 2
              309.25     771.00
              319.25     771.00
              379.25     756.00

         4    2 Levee 3
              302.50     762.50
              304.00     762.50
              304.25     764.75
              313.75     764.75
              360.00     756.00

         5    3 Blanket
              100.00     740.00
              203.00     740.00
              360.00     756.00
              500.00     756.00

         6    4 Twall
              302.50     762.50
              302.75     766.00
              307.75     766.00
              308.00     770.00
              308.25     775.00
              309.00     775.00
              309.25     771.00
              309.50     766.00
              313.50     766.00
              313.75     764.75

         7    5 Aquifer
              100.00     725.00
              203.00     725.00
              360.00     733.00
              500.00     733.00

         1    1 Riprap and Bedding
              196.00     740.00
              301.00     770.00
              308.00     770.00


MATERIAL PROPERTIES
     1 Riprap and Bedding
          125.00 Unit Weight
```

```
          Conventional Shear
                 .00     35.00
          Piezometric Line
          1
     2 Levee
          120.00 Unit Weight
          Conventional Shear
                 .00     29.00
          Piezometric Line
          1
     3 Blanket
          115.00 Unit Weight
          Conventional Shear
                 .00     26.00
          Interpolate Pore Water Pressure
     4 Twall
          150.00 Unit Weight
          Very Strong
     5 Aquifer
          120.00 Unit Weight
          Conventional Shear
                 .00     32.00
          Piezometric Line
          2
     6 Berm
          115.00 Unit Weight
          Conventional Shear
                 .00     29.00
          Piezometric Line
          1

PIEZOMETRIC LINES
         1      62.40 Piezometric Line 1
               100.00    775.00
               308.25    775.00
               309.25    771.00
               313.75    764.75
               360.00    756.00
               600.00    756.00

         2      62.40 Piezometric Line 2
                85.00    773.00
               360.00    768.40
               460.00    766.80
               560.00    765.40
               600.00    764.70


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
         120.00     740.00    2184.00         3
         120.00     733.00    2545.10         3
         120.00     725.00    2957.80         3
         160.00     740.00    2184.00         3
         160.00     733.00    2524.70         3
         160.00     725.00    2914.00         3
         203.00     740.00    2184.00         3
         203.00     733.00    2650.80         3
         203.00     725.00    2870.40         3
```

```
        240.00      743.80    1864.80           3
        240.00      735.00    2307.20           3
        240.00      726.90    2714.40           3
        270.00      746.80    1634.40           3
        270.00      735.00    2251.00           3
        270.00      728.40    2595.80           3
        308.25      750.70    1516.30           3
        308.25      740.00    1996.50           3
        308.25      730.40    2427.40           3
        313.75      751.30     839.30           3
        313.75      740.00    1696.00           3
        313.75      730.60    2408.60           3
        336.00      753.60     433.70           3
        336.00      740.00    1603.50           3
        336.00      731.80    2308.80           3
        360.00      756.00        .00           3
        360.00      744.00    1152.50           3
        360.00      733.00    2209.00           3
        400.00      756.00        .00           3
        400.00      744.00    1038.50           3
        400.00      733.00    2171.50           3
        440.00      756.00        .00           3
        440.00      744.00    1110.20           3
        440.00      733.00    2127.80           3
        480.00      756.00        .00           3
        480.00      744.00    1090.60           3
        480.00      733.00    2090.40           3
        520.00      756.00        .00           3
        520.00      744.00    1074.40           3
        520.00      733.00    2059.20           3
        560.00      756.00        .00           3
        560.00      744.00    1054.90           3
        560.00      733.00    2021.80           3
        600.00      756.00        .00           3
        600.00      744.00    1032.10           3
        600.00      733.00    1978.10           3


ANALYSIS/COMPUTATION
     Circular Search 1
        364.00      816.00       1.00      670.00
     Radius
          82.00
SINgle-stage Computations
SAVe n most
 150
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

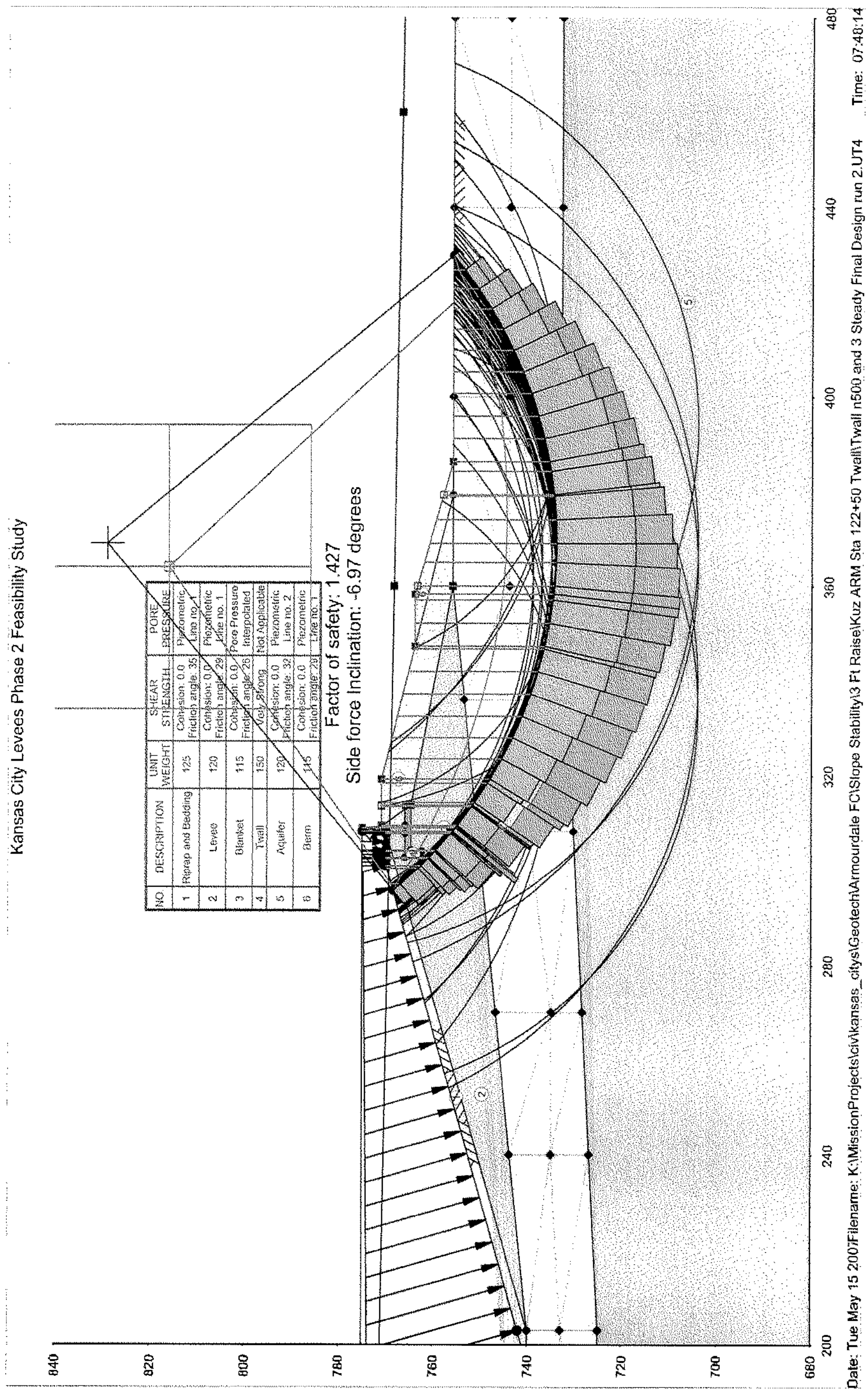

Kansas City Levees Phase 2 Feasibility Study
NO
DESCRIPTION
UNIT WEIGHT
SHEAR STRENGTH
PORE PRESSURE
1
Riprap and Bedding
125
Cohesion: 0.0
Friction angle: 35
Piezometric
Line no. 1
2
Levee
120
Cohesion: 0.0
Friction angle: 29
Piezometric
Line no. 1
3
Blanket
115
Cohesion: 0.0
Friction angle: 26
Pore Pressure
Interpolated
4
Twall
150
Very Strong
Not Applicable
5
Aquifer
120
Cohesion: 0.0
Friction angle: 32
Piezometric
Line no. 2
6
Berm
115
Cohesion: 0.0
Friction angle: 29
Piezometric
Line no. 1
Factor of safety: 1.427
Side force inclination: -6.97 degrees
840
820
800
780
760
740
720
700
680
200
240
280
320
360
400
440
480
Date: Tue May 15 2007
Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 122+50 Twall\Twall n500 and 3 Steady Final Design run 2.UT4
Time: 07:48:14

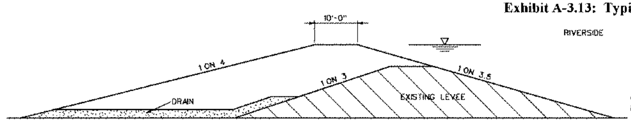
Exhibit A-3.12
Proposed Levee Raise Summary

Armourdale Levee Unit
Phase 2 Feasibility Study
4/26/2007 rev 2

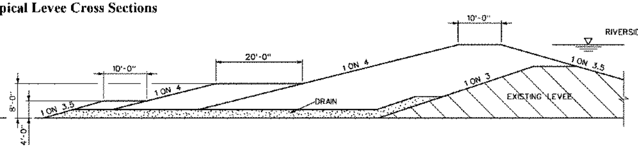
Station	Existing Levee Crest Elevation (ft)	Existing Landside Elevation (ft)	Existing Levee Landside Height (ft)	n500+3 Raise (ft)	First Berm: Berm Height (ft)	First Berm: Berm Width (ft)	Second Berm: Berm Height	Second Berm: Berm Width	Proposed Landside Height (ft)	Distance from Exist. Centerline to Exist. Toe (ft)	Proposed Section Number	Landside: T-Wall Distance from Exist. Centerline to Proposed Toe (ft)	Landside: Levee Distance from Exist. Centerline to Proposed Toe (ft)
3+25 UE to 10+00 UE	774.10	765.0	9.10	4.10	6	12			13.20	32.3	3		**47.3**
10+00 UE to 16+48 UE	774.10	765.0	9.10	4.00					13.10	32.3	Floodwall		
16+48 UE To 15+00	773.95		high ground	4.05						5.0	1		**19.2**
15+00 To 20+00	773.95	770.0	3.95	4.05					8.00	15.9	1		**51.2**
20+00 to 25+00	773.90	768.0	5.90	4.05					9.95	22.7	1		**59.0**
25+00 to 30+00	773.80	768.0	5.80	4.10					9.90	22.4	1		**58.9**
30+00 to 35+00	773.75	768.0	5.75	4.10					9.85	22.3	1		**58.8**
35+00 to 40+00	773.65	760.0	13.65	4.15	8	20	4	10	17.80	45.9	2		**118.7**
40+00 to 42+50	773.60	760.0	13.60	4.20	8	20	4	10	17.80	45.8	2		**118.9**
42+50 to 45+00	773.50	760.0	13.50	4.30	8	10			17.80	45.5	7	**70.5**	**101.3**
45+00 to 50+00	773.45	760.0	13.45	4.30	8	10			17.75	45.4	7	**70.3**	**101.1**
50+00 to 55+00	773.40	762.0	11.40	4.30	6	8			15.70	39.2	6	**61.1**	**82.8**
55+00 to 58+00	773.35	762.0	11.35	4.30	6	8			15.65	39.1	6	**60.9**	**82.7**
58+00 to 60+40	773.25	762.0	11.25	4.30					15.55	38.8	?		
60+40 to 65+00	773.25	762.0	11.25	4.15					15.40	38.8	?		**existing toe**
65+00 to 70+00	773.15	760.0	13.15	4.20					17.35	44.4	?		**existing toe**
70+00 to 74+00	772.85	758.0	14.85	4.35					19.20	49.6	?		**existing toe**
74+00 to 77+78											Floodwall		
77+78 to 81+00	772.25	760.0	12.25	4.80	8	10			17.05	41.8	7	**65.5**	**98.0**
81+00 to 86+00	772.10	760.0	12.10	4.80					16.90	41.3	1		**89.4**
86+00 to 90+00	771.75	760.0	11.75	4.80					16.55	40.3	1		**88.0**
90+00 to 95+00	771.50	760.0	11.50	4.80					16.30	39.5	1		**87.0**
95+00 to 96+00	771.15	760.0	11.15	4.85					16.00	38.4	1		**86.0**
96+00 to 100+00	771.00	760.0	11.00	4.90	6	8			15.90	38.0	6	**59.5**	**85.8**
100+00 to 105+00	770.60	760.0	10.60	5.20	6	8			15.80	36.8	6	**57.9**	**86.4**
105+00 to 110+00	770.45	762.0	8.45	5.10					13.55	30.4	1		**77.1**
110+00 to 115+00	770.25	760.0	10.25	5.05					15.30	35.8	1		**83.9**
115+00 to 120+00	770.10	756.0	14.10	4.95	8	20	4	10	19.05	47.3	2		**126.5**
120+00 to 125+00	769.90	756.0	13.90	4.90	8	20	4	10	18.80	46.7	2		**125.4**
125+00 to 129+00	769.70	756.0	13.70	4.85	8	20	4	10	18.55	46.1	2		**124.2**
129+00 to 130+00	769.60	758.0	11.60	4.75					16.35	39.8	1		**87.0**
130+00 to 135+00	769.55	764.0	5.55	4.75					10.30	21.6	5	**32.7**	**62.8**
135+00 to 140+00	769.40	764.0	5.40	4.75					10.15	21.2	5	**32.1**	**62.2**
140+00 to 145+00	769.20	764.0	5.20	4.75					9.95	20.6	5	**31.3**	**61.4**
145+00 to 150+00	769.00	764.0	5.00	4.75					9.75	20.0	5	**30.5**	**60.6**
150+00 to 156+50	768.85	764.0	4.85	4.75					9.60	19.6	5	**29.9**	**60.0**
156+50 to 160+00	768.65	764.0	4.65	4.70					9.35	18.9	1		**58.8**
160+00 to 165+00	768.55	764.0	4.55	4.65					9.20	18.6	1		**58.1**
165+00 to 170+00	768.40	764.0	4.40	4.65					9.05	18.2	1		**57.5**
170+00 to 175+00	768.25	764.0	4.25	4.60					8.85	17.8	1		**56.5**
175+00 to 180+00	768.05	764.0	4.05	4.55					8.60	17.1	1		**55.3**
180+00 to 185+00	767.85	764.0	3.85	4.40					8.25	16.6	1		**53.4**
185+00 to 189+00	767.65	752.0	15.65	4.30	8	20	4	10	19.95	51.9	2		**127.9**
189+00 to 197+00											Floodwall		
197+00 to 200+00	767.05	754.0	13.05	4.30					17.35	44.1	1		**89.4**
200+00 to 205+00	767.00	754.0	13.00	4.00					17.00	44.0	1		**87.0**
205+00 to 206+12.43	766.50	756.0	10.50	4.15					14.65	36.5	1		**78.1**
212+00 to 225+44											Floodwall		
225+44 to 228+00											Stoplog Gap		
228+00 to 230+00	765.40	750.0	15.40	4.20	10	15			19.60	51.2	?	**82.1**	**113.1**
230+00 to 235+00	765.30	750.0	15.30	4.10	10	15			19.40	50.9	?	**81.7**	**112.0**
235+00 to 240+00	765.00	748.0	17.00	4.00	10	15	5	15	21.00	56.0	?	**103.5**	**130.5**
240+00 to 245+00	764.66	748.0	16.66	3.95	10	15	5	15	20.61	55.0	?	**102.1**	**128.8**
245+00 to 246+50	764.35	748.0	16.35	3.80	8	20	4	10	20.15	54.1	2		**126.9**
246+50 to 248+50											Floodwall		
248+50 to 249+30	763.90	756.0	7.90	3.95					11.85		1		**66.2**
249+30 to 250+60											Floodwall		
250+60 to 255+00	763.55	750.0	13.55	4.15	8	10			17.70	45.6	?	**70.7**	**100.3**
255+00 to 257+65	763.45	750.0	13.45	4.15	8	10			17.60	45.4	?	**70.3**	**99.9**
257+65 to 295+50											Floodwall		
295+50 to 300+00	761.65	744.0	17.65	1.80	8	20	4	10	19.45	57.9	2		**117.1**
300+00 to 302+58	761.65	744.0	17.65	1.35	8	20	4	10	19.00	57.9	2		**113.7**
302+58 to 305+00	761.65	746.0	15.65	1.10					16.75	51.9	1		**75.8**
305+00 to 310+00	760.65	748.0	12.65	1.80					14.45	42.9	1		**69.1**
310+00 to 315+00	760.65	748.0	12.65	1.40					14.05	42.9	1		**68.1**
315+00 to 320+00	760.65	752.0	8.65	1.30					9.95	30.9	1		**49.3**
320+00 to 322+85.41	760.65	756.0	4.65	1.20					5.85	18.9	1		**32.6**

Notes:
1. Distance from existing centerline to the existing toe assumes a 1V on 3H landside slope. This is not always the case.
2. Station 185+00 to 189+00, assumed a berm for this work. Should be analyzed.
3. Computed levee proposed toe for the proposed T-wall reaches assume no stability berms required for proposed raises under 17 feet, and may be incorrect in some cases. Numbers shown is the minimum required real estate.
4. Computed levee proposed toe for the proposed T-wall reaches assume stability berms equal to those required for the T-wall for proposed raises over 17 feet, and may be incorrect in some cases.
Numbers shown indicate minimum real estate needs.

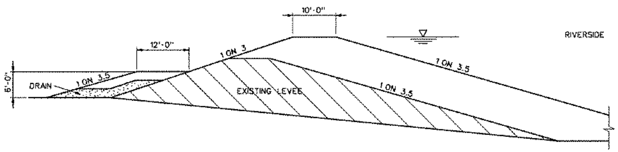
Exhibit A-3.13: Typical Levee Cross Sections



SECTION 1 LANDSIDE EARTH FILL RAISE (STA. 90+00±)
SCALE: 1" = 10'-0"



SECTION 2 LANDSIDE EARTH FILL RAISE (STA. 246+00±)
SCALE: 1" = 10'-0"

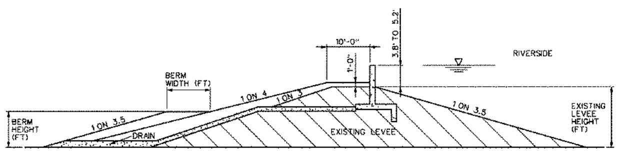


SECTION 3 RIVERSIDE EARTH FILL RAISE (STA. 7+00 UE±)
SCALE: 1" = 10'-0"

SECTION 4 -- NOT USED
NO SCALE

NOTES:

1. UNDERSEEPAGE FEATURES ARE NOT SHOWN.



SECTION 5 THRU 9 (SEE TABLE) WALL ON EXISTING LEVEE
SCALE: 1" = 10'-0"

	EXISTING LEVEE HEIGHT	SECTION	SINGLE BERM		SECOND BERM**	
			BERM HEIGHT	BERM WIDTH	BERM HEIGHT	BERM WIDTH
	< 10	5	N/A	N/A	N/A	N/A
*	10-12	6	6	8	N/A	N/A
	12-14	7	8	10	N/A	N/A
*	14-16	8	10	15	N/A	N/A
*	16-18	9	10	15	5	15

* SECTION REQUIREMENTS ESTIMATED FROM TWO SECTIONS ANALYZED
** SECOND BERM NOT SHOWN IN TYPICAL CROSS SECTION

US Army Corps of Engineers
Kansas City District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: R. KUZNIAKOWSKI
Drawn by: J. PAULSON
Checked by: M. CORKILL
Submitted by: D. MATHENS
Date: MAY 2007
File no. X
Plot scale: 120
CADD File Name: KC_AU_CI.DGN

KANSAS CITY
ARMOURDALE UNIT

TYPICAL CROSS SECTIONS

Sheet Reference Number: C1

K:\MISSIONPROJECTS\CIVIL\KANSAS_CITYS\STRUCTURES\PHASE 2

EXHIBIT A-3.14: Armourmdale Unit Underseepage Summary

Armourdale Levee Kansas City Phase 2 Feasibility
Thursday, December 28, 2006
Levee Foundation Information - N500+3, Water to Top of Levee

Blanket Unit Weight = 115.0 pcf (saturated)

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio River Side	Permeability Ratio Land Side	Impervious Blanket Thickness (ft) River Side	Impervious Blanket Thickness (ft) Levee	Impervious Blanket Thickness (ft) Land Side	Pervious Blanket Thickness (ft) d	Seepage Length (ft) River Side L_1	Seepage Length (ft) Levee L_2	Seepage Length (ft) Land Side L_3	Factor c_1	Factor c	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_0	Computed Hydraulic Gradient i	Critical Hydraulic Gradient i_c	Factor of Safety for Piping	Remarks
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EXHIBIT A-3.15: Special Features Analyzed for Underseepage

								Pressures on Base of Blanket			Pressures on Base of Feature					
Building or Feature	Station Start	Station Stop	Distance From Seepage Entrance	Top of Ground	Bottom of Blanket	Design Feature Elevation	Design Feature Elevation Source	$h_{o\text{-}allow}$ to protect blanket (ft)	Existing Conditions - '62-MOD Calculations h_p (ft)	Existing Conditions - Feasibility Calculations h_p (ft)	N500+3 h_p (ft)	Hydrostatic Pressure Head at Feature Elevation (ft)	Existing Pressure Head at Feature Elevation (ft)	N500+3 Pressure Head at Feature Elevation (ft)	Increase in Pressure at Feature Elevation between Existing and N500+3 (psf)	Remedial Measure for Blanket Underseepage
Proctor and Gamble	5700	6200	500	762	735	755	DM3	10.5	7.0	7.0	9.7	7.0	8.8	9.5	0.7	None Required
Inland Container	6800	6800	850 (around wall)	762	743	754	DM3	5.8	9.2	9.2	5.5	8.0	11.9	10.3	-1.6	Cut-Off
Mill Street Pump Station Ditch	15700	15700	400	760	730	752	Topo	-	-	-	-	-	-	-	-	Fill with impervious
Midwest Cold Storage	19200	19700	200	752	728	742	Assumed	7.4	7.4	9.6	7.5	10.0	13.1	13.1	0.0	Wells
Kansas Fish and Oyster	19700	20000	400	752	728	745	DM3	9.0	3.7	5.9	5.8	7.0	8.1	8.7	0.6	Wells
Schock Truck and Leasing	21400	21400	350	752	728	742	Assumed	7.4	4.9	7.1	7.4	10.0	12.0	13.1	1.0	Wells
Jones Store Distributing	21700	21700	300	752	722	742	Assumed	10.5	6.3	8.5	9.0	10.0	12.1	13.0	0.9	Wells
Glass Co - Prime Investments	21800	22000	250	750	722	740	Assumed	9.5	8.3	10.5	8.9	10.0	13.0	13.2	0.2	Wells
Overland Park Bank	22500	22500	270	748	722	738	Assumed	8.4	8.0	10.2	8.8	10.0	13.1	12.6	-0.5	Wells
Sambol Meat Packing	22900	23050	250	748	725	742	DM3	9.0	7.4	9.6	8.0	6.0	7.9	8.1	0.2	Wells
Selco	23100	23500	250	749	730	748	DM3	9.5	8.4	10.6	8.1	1.0	1.4	1.4	0.0	Wells
Selco	23500	23700	350	749	730	748	DM3	9.5	7.8	10.0	7.5	1.0	1.4	1.4	0.0	Wells
PBI Gordon Co	28300	28600	200	746	717	743	DM3	13.7	12.2	12.2	14.0	3.0	4.3	4.4	0.2	None Required

*Assuming buildings built since 2000, including Pensky, Pitney Bowes, and USF Distribution do not have basements
*All building basement elevations were assumed to be 10 feet below grade if not found in DM-3
*Any building not specificially called out in table was not analyzed
*$h_{o\text{-}allow}$ is calculated using a $FS_i = 1.60$
*h_p is excess head measured from top of surrounding ground"
*Assumed hydrostatic water pressures act on top of blanket at design feature elevation with water elevation at the top of the surrounding ground
*All Feature Elevations should be verified during final design.
*Existing Conditions 1962-MOD calculations were estimated by subtracting 2.2 feet (AVG Difference in calculations at checkpoints) from the Existing Condtions Feasibility Calculations

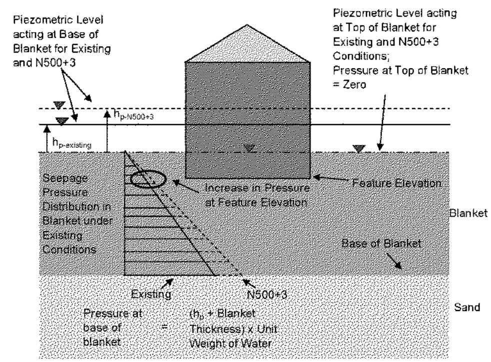


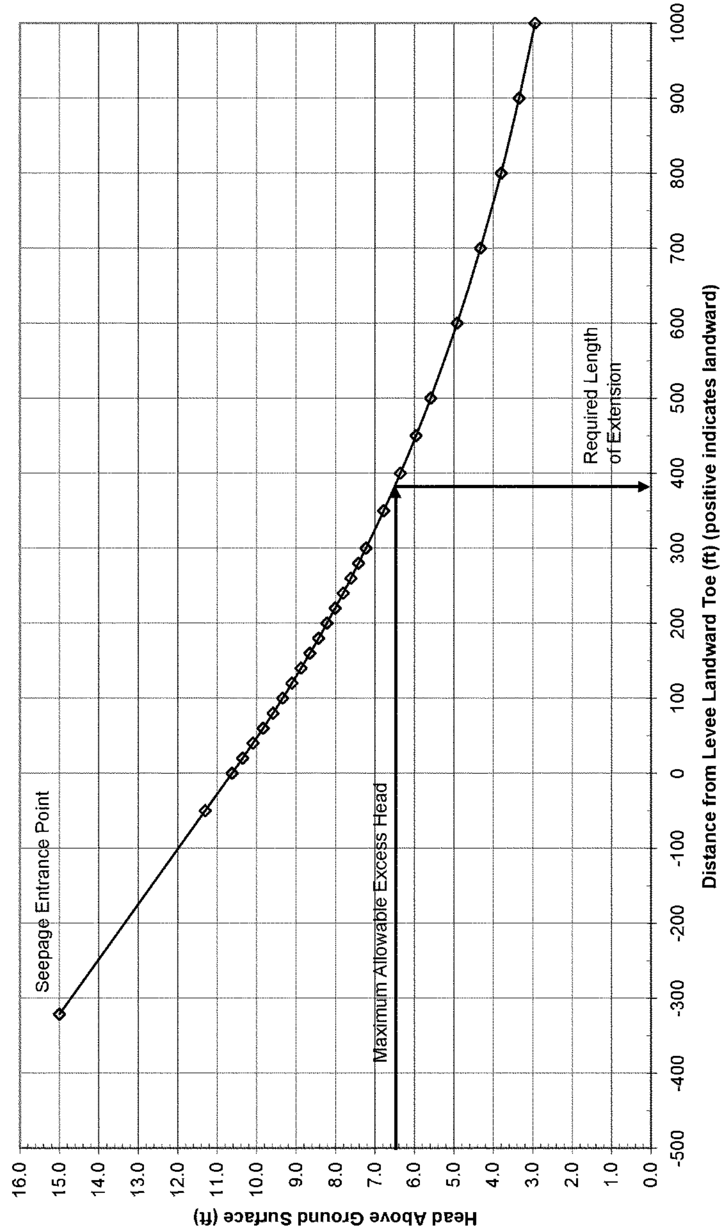
Exhibit A-3.16
Underseepage Comparison Calculations

			1962 Modification Calculations			Feasibility Study Calculations			1962 minus Feasibility		
Checkpoint	Station	Seepage Distance	Initial HGL	Drawdown (ft)	HGL	Initial HGL	Drawdown (ft)	HGL	Initial HGL	Drawdown (ft)	HGL
C-C	194+50	235	762.5	4.3	758.3	762.8	2.3	760.5	-0.3	2.0	-2.3
C-D	193+75	400	760.5	4.3	756.2	760.4	2.3	758.1	0.1	2.0	-1.9
C-E	197+00	250	762.5	4.5	758.0	762.6	2.5	760.1	-0.1	2.0	-2.1
C-I	199+30	440	760.0	4.7	755.3	759.9	2.4	757.5	0.1	2.3	-2.2
C-J	204+30	350	760.5	4.5	756.0	761.1	2.2	758.9	-0.6	2.3	-2.9
C-K	212+70	270	762.5	4.6	757.9	761.9	1.8	760.1	0.6	2.8	-2.2
C-L	215+50	270	762.0	3.9	758.1	761.9	1.2	760.7	0.1	2.7	-2.6
C-M	215+50	400	758.5	4.3	754.2	760.2	2.6	757.6	-1.7	1.7	-3.4
C-O	220+20	325	758.0	5.9	752.1	759.3	3.8	755.5	-1.3	2.1	-3.4
C-P	220+70	350	758.0	5.6	752.4	758.9	3.4	755.5	-0.9	2.2	-3.1
C-Q	221+50	240	761.0	5.8	755.3	760.9	5.0	755.9	0.1	0.8	-0.6
C-R	222+00	430	757.9	5.3	752.6	757.6	4.0	753.6	0.3	1.3	-1.0
C-S	222+50	230	761.5	5.6	755.9	761.1	4.6	756.5	0.4	1.0	-0.6
C-T	225+20	240	761.0	4.2	756.8	760.9	2.1	758.8	0.1	2.1	-2.0
C-U	225+00	365	759.5	4.6	754.9	758.1	1.6	756.5	1.4	3.0	-1.6
C-V	225+00	500	757.5	4.6	752.9	756.2	1.3	754.9	1.3	3.3	-2.0
C-W	229+40	280	759.0	4.7	754.3	759.6	2.6	757.0	-0.6	2.1	-2.7
C-X	231+50	200	760.5	4.1	756.4	761.2	2.2	759.0	-0.7	1.9	-2.6
C-Y	231+80	330	758.5	4.4	754.2	758.2	1.1	757.1	0.3	3.3	-3.0
C-A'	234+70	420	759.0	4.2	754.8	756.9	1.4	755.5	2.1	2.8	-0.7
C-C'	236+70	280	759.0	4.4	754.6	759	1.9	757.1	0.0	2.5	-2.5
C-D'	240+80	230	760.0	7.8	752.2	759.9	5.6	754.3	0.1	2.2	-2.1
C-E'	241+00	400	757.5	6.6	750.9	757.2	4.4	752.8	0.3	2.2	-1.9
C-H'	242+75	250	759.0	7.8	751.2	759.6	5.5	754.1	-0.6	2.3	-2.9
C-I'	243+50	350	758.0	6.9	751.1	757.9	4.6	753.3	0.1	2.3	-2.2
C-J'	245+20	210	760.0	7.2	752.8	760.3	5.1	755.2	-0.3	2.1	-2.4
C-K'	247+20	250	760.0	5.1	754.9	759.6	2.8	756.8	0.4	2.3	-1.9
C-M'	249+20	320	759.0	4.2	754.8	758.4	1.6	756.8	0.6	2.6	-2.0

EXHIBIT A-3.17

Hydraulic Grade Line for Cutoff Wall Extension

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 60+00 to 66+00
N500+3



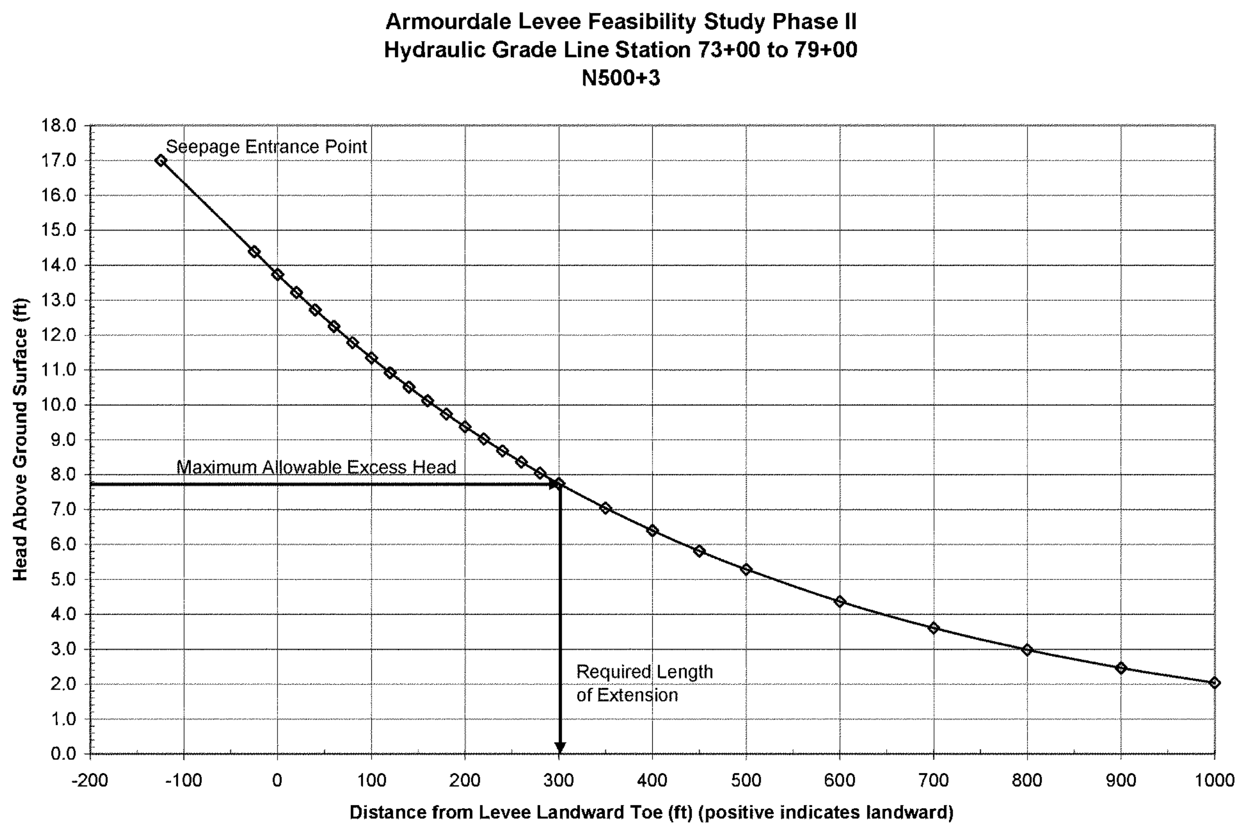

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 73+00 to 79+00
N500+3
Seepage Entrance Point
Maximum Allowable Excess Head
Required Length
of Extension
Head Above Ground Surface (ft)
Distance from Levee Landward Toe (ft) (positive indicates landward)
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-100
0
100
200
300
400
500
600
700
800
900
1000

Exhibit A-3.18

Proposed Relief Well System, Station 190+00 to 254+00

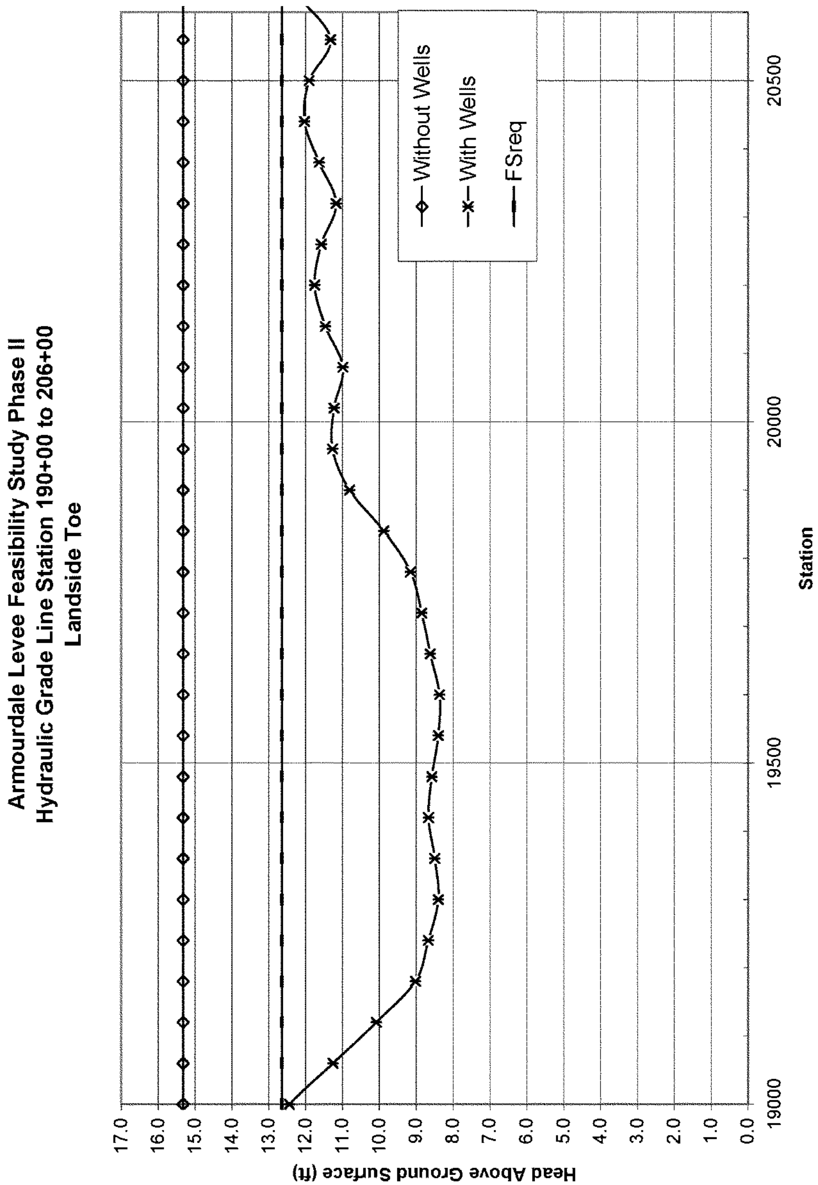
Status	Well	Distance From Seepage Entrance	Station	Station Designation	Discharge Elevation (ft)	Flow (cfs)
Existing	1	230	19075	BK	751.0	0.97
Proposed	25	220	19150	BK	752.0	0.77
Proposed	26	220	19175	BK	752.0	0.74
Proposed	27	220	19200	BK	752.0	0.73
Proposed	28	220	19250	BK	752.0	0.72
Proposed	29	220	19300	BK	752.0	0.70
Existing	2	230	19325	BK	749.0	0.82
Proposed	30	220	19350	BK	752.0	0.70
Proposed	31	220	19430	BK	752.0	0.74
Proposed	32	220	19490	BK	752.0	0.73
Proposed	33	220	19550	BK	752.0	0.72
Existing	3	240	19580	BK	747.5	0.91
Proposed	34	220	19610	BK	752.0	0.73
Proposed	35	220	19670	BK	752.0	0.77
Proposed	36	220	19730	BK	752.0	0.80
Proposed	37	220	19780	BK	752.0	0.84
Existing	4	230	19830	BK	746.0	1.24
Existing	5	230	20080	BK	743.5	1.81
Existing	6	230	20325	BK	742.5	2.09
Existing	7	220	20574	BK	740.5	2.67
Existing	8	220	21415	AH	739.0	2.72
Proposed	38	180	21680	AH	752.0	1.49
Proposed	39	180	21800	AH	748.0	1.29
Existing	9	220	21916	AH	736.5	3.07
Existing	10	230	22016	AH	736.0	2.02
Existing	11	240	22209	AH	734.5	2.12
Existing	12	240	22407	AH	733.5	2.66
Proposed	40	180	22450	AH	748.0	1.43
Proposed	41	180	22500	AH	748.0	1.41
Proposed	42	180	22600	AH	748.0	1.45
Proposed	43	180	22700	AH	748.0	1.50
Existing	13	230	22838	AH	731.0	2.97
Existing	14	240	23053	AH	730.0	3.08
Existing	15	200	23238	AH	739.5	2.25
Existing	16	200	23396	AH	741.5	1.87
Proposed	44	180	23460	AH	748.0	1.31
Proposed	45	180	23500	AH	748.0	1.30
Proposed	46	180	23560	AH	749.0	1.29
Proposed	47	180	23680	AH	749.0	1.43
Existing	17	220	23795	AH	743.5	2.02
Proposed	48	180	23900	AH	749.0	1.17
Existing	18	180	23995	AH	744.5	1.51
Existing	19	180	24095	AH	745.0	1.47
Existing	20	180	24189	AH	745.5	1.43
Existing	21	180	24290	AH	746.5	2.02
Existing	22	180	24396	AH	747.0	1.75
Proposed	49	180	24460	AH	749.0	1.49
Existing	23	180	24515	AH	747.5	1.65
Existing	24	190	24635	AH	749.0	1.58
Proposed	50	180	24800	AH	751.0	1.44
Proposed	51	180	24950	AH	751.0	1.32
Proposed	52	180	25075	AH	751.0	1.13
Proposed	53	180	25100	AH	752.0	1.01
Proposed	54	180	25130	AH	752.0	1.01
Proposed	55	180	25170	AH	752.0	1.06
Proposed	56	180	25210	AH	752.0	1.15
Proposed	57	180	25240	AH	752.0	0.78
Proposed	58	180	25280	AH	752.0	0.79
Proposed	59	180	25320	AH	752.0	0.80
Proposed	60	180	25360	AH	752.0	0.81
Proposed	61	180	25390	AH	752.0	0.82
Proposed	62	180	25420	AH	752.0	0.85
Proposed	63	180	25440	AH	752.0	0.90

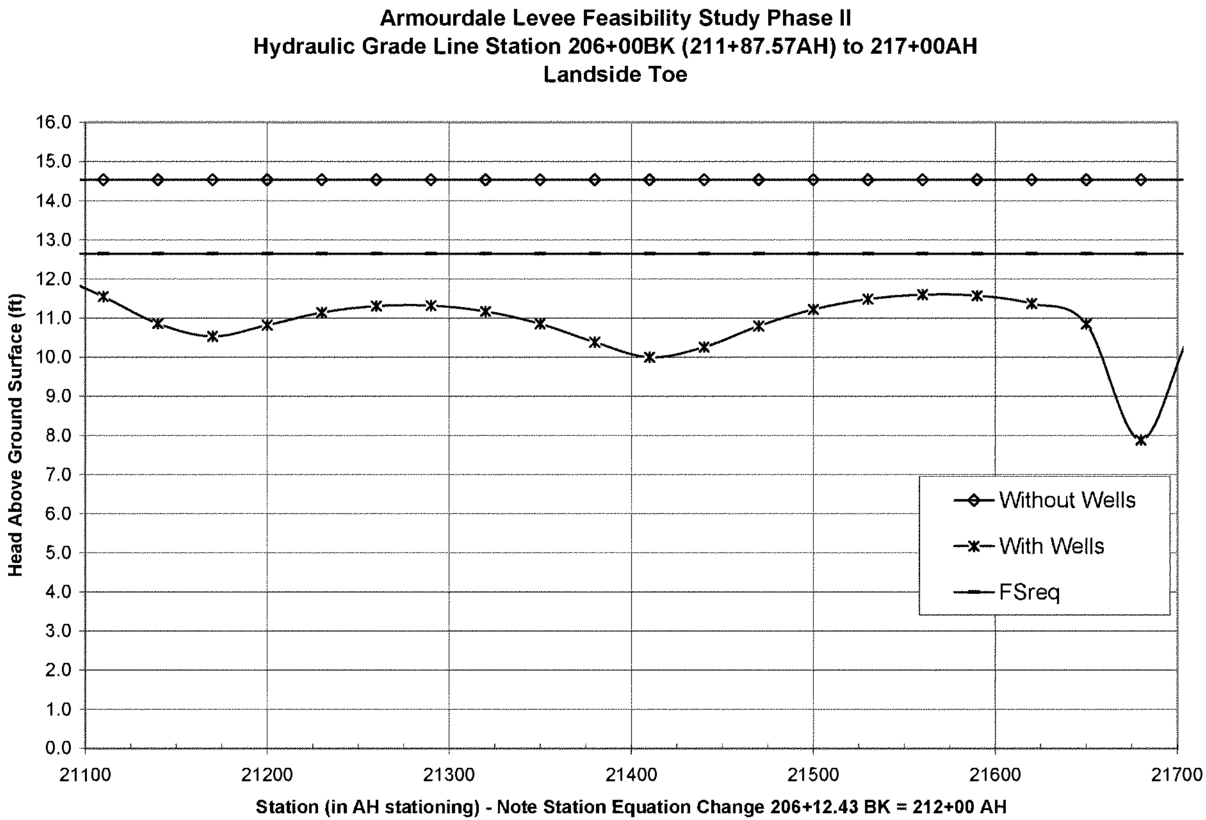
*Relief well flows shown are expected flows. The flows have been reduced by 80% in the calculations to account for future well efficiency reduction

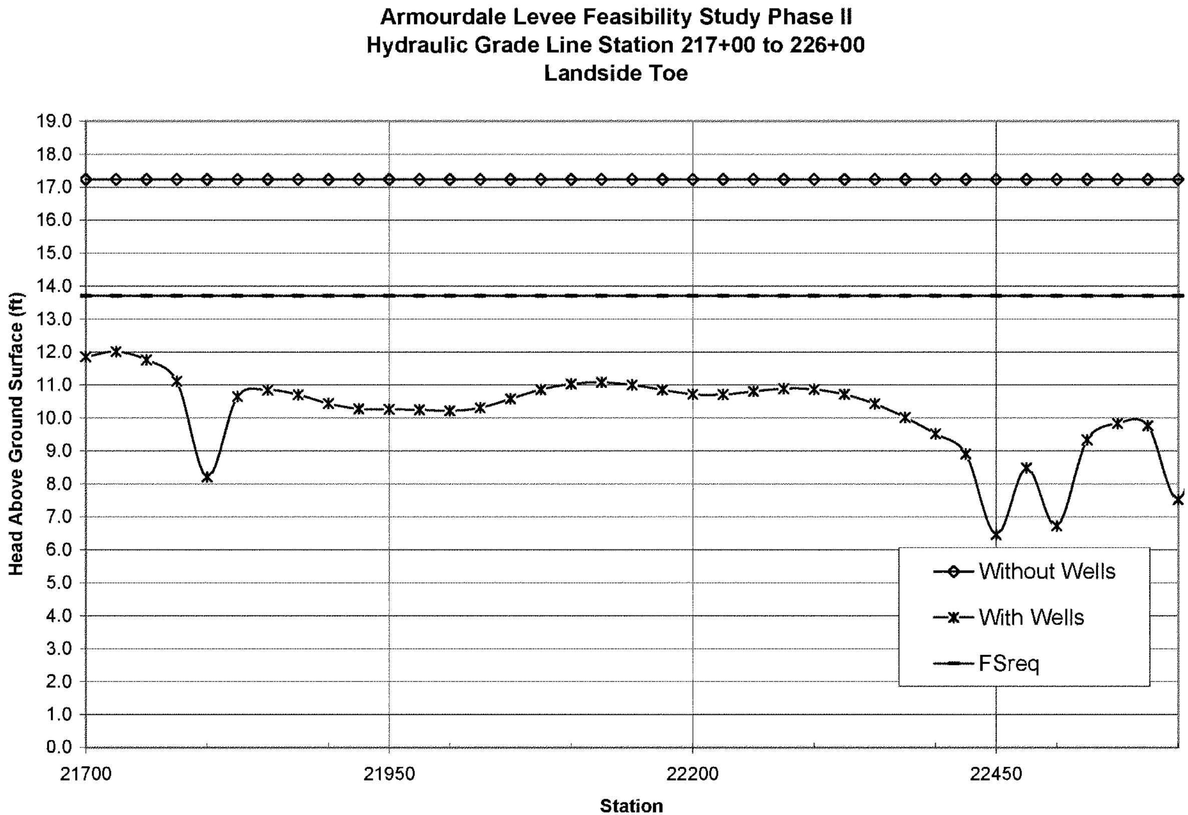
*Seepage engrance is assumed to be at the elevation of the bottom of the riverside blanket

EXHIBIT A-3.19

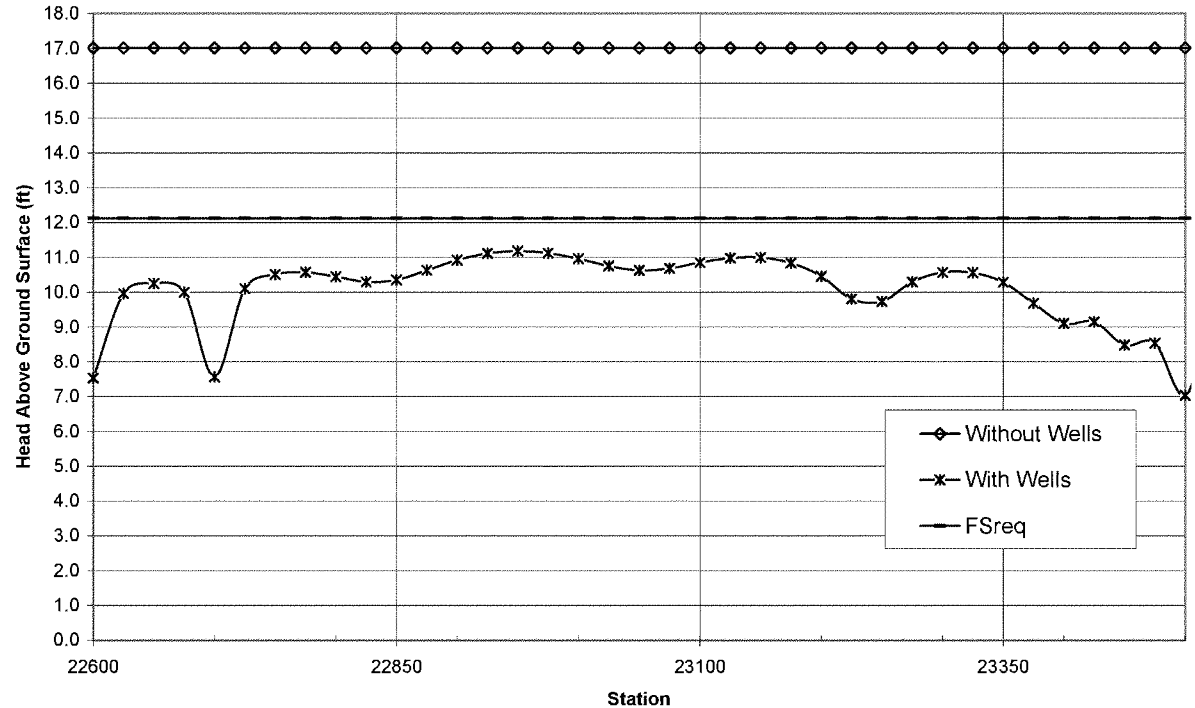
Computed Excess Head at Land Side Toe

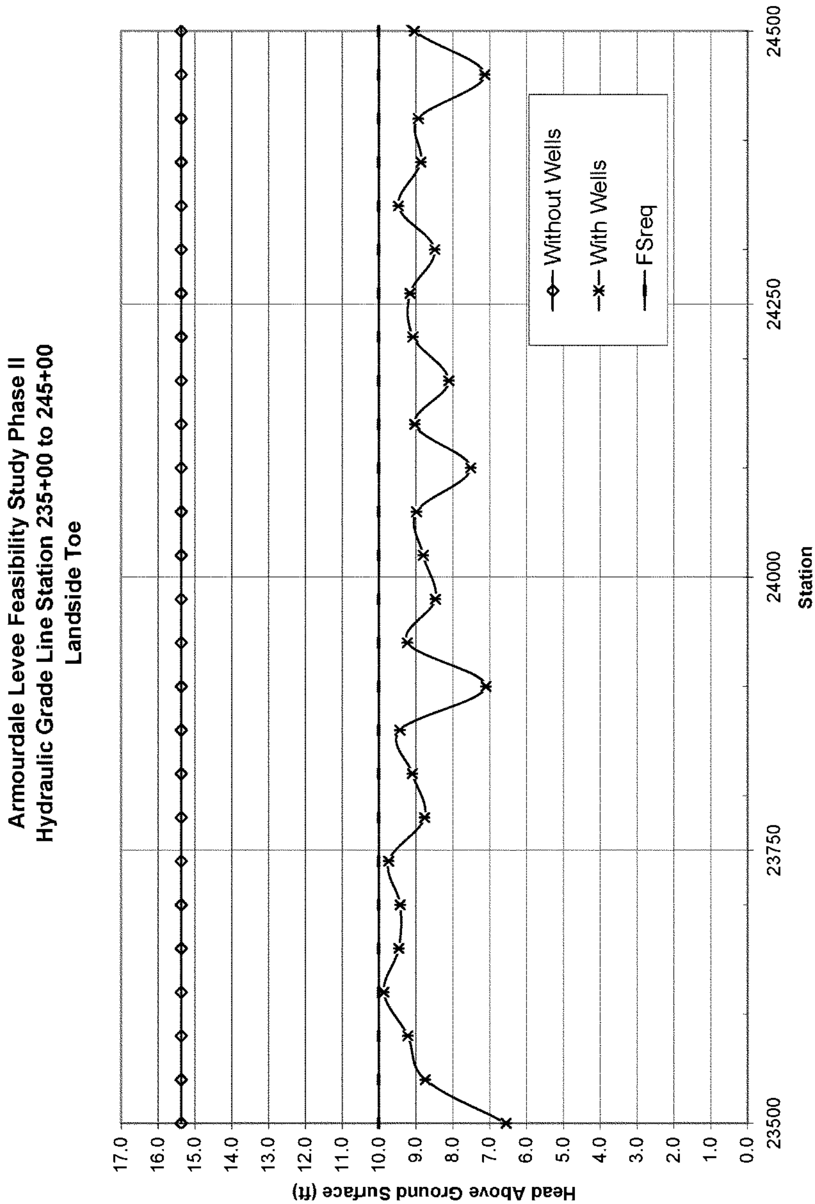

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 190+00 to 206+00
Landside Toe
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
19000
19500
20000
20500
Station
Without Wells
With Wells
FSreq


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 206+00BK (211+87.57AH) to 217+00AH
Landside Toe
Head Above Ground Surface (ft)
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
21100
21200
21300
21400
21500
21600
21700
Without Wells
With Wells
FSreq
Station (in AH stationing) - Note Station Equation Change 206+12.43 BK = 212+00 AH

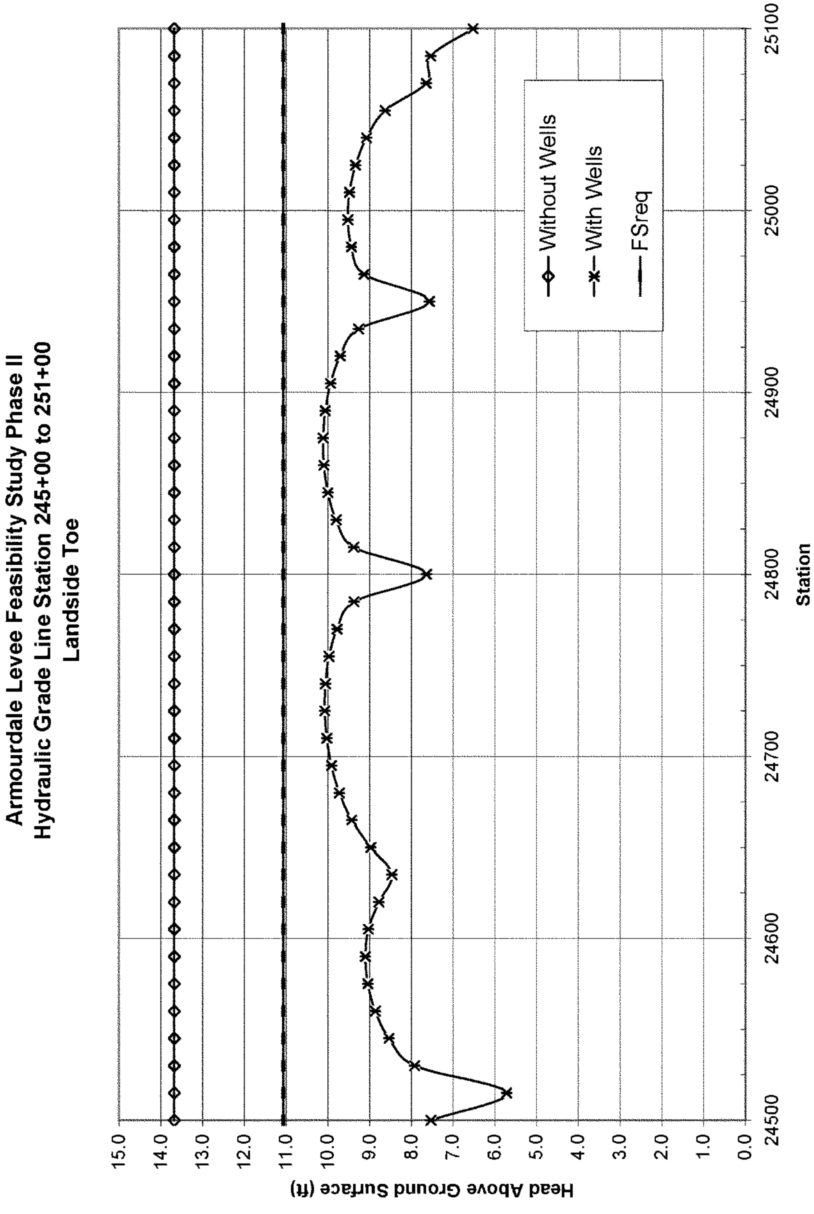

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 217+00 to 226+00
Landside Toe
Head Above Ground Surface (ft)
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
21700
21950
22200
22450
Station
Without Wells
With Wells
FSreq

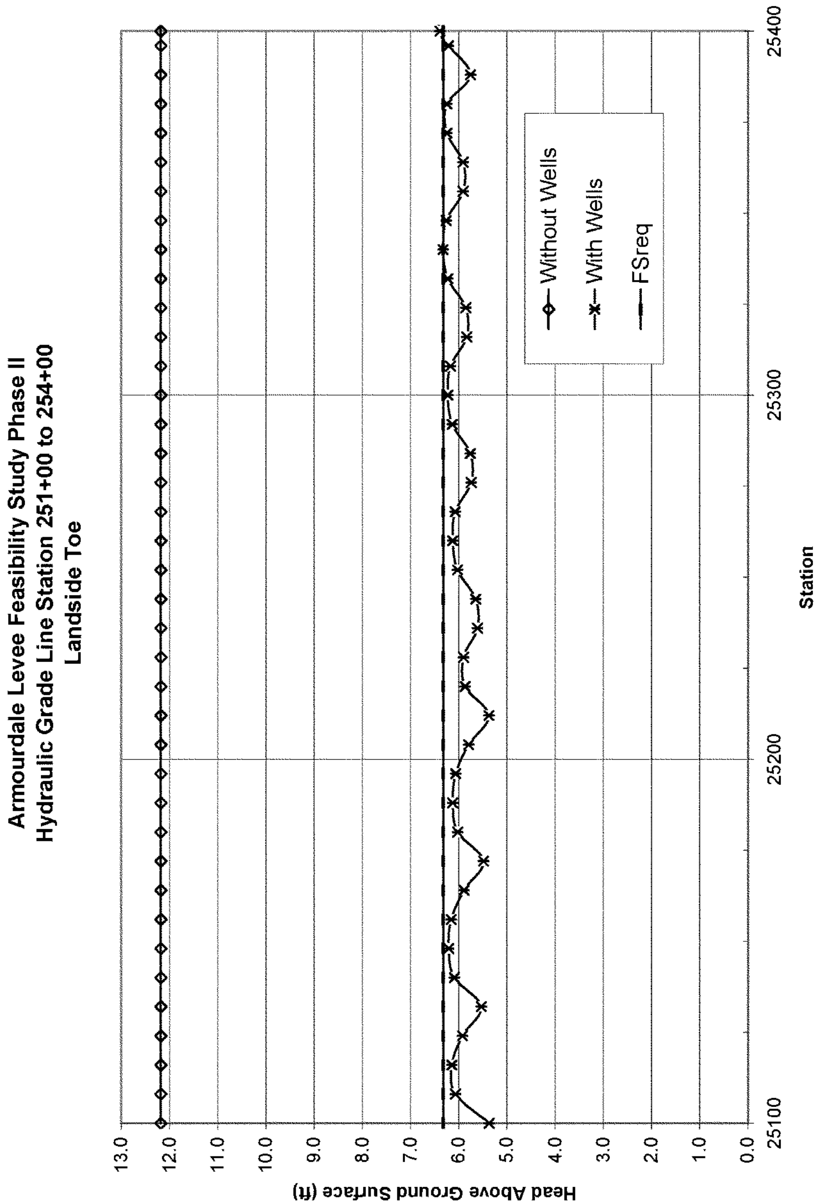
Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 226+00 to 235+00
Landside Toe





Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 235+00 to 245+00
Landside Toe
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
23500
23750
24000
24250
24500
Station
Without Wells
With Wells
FSreq


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 245+00 to 251+00
Landside Toe
Head Above Ground Surface (ft)
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
24500
24600
24700
24800
24900
25000
25100
Station
Without Wells
With Wells
FSreq

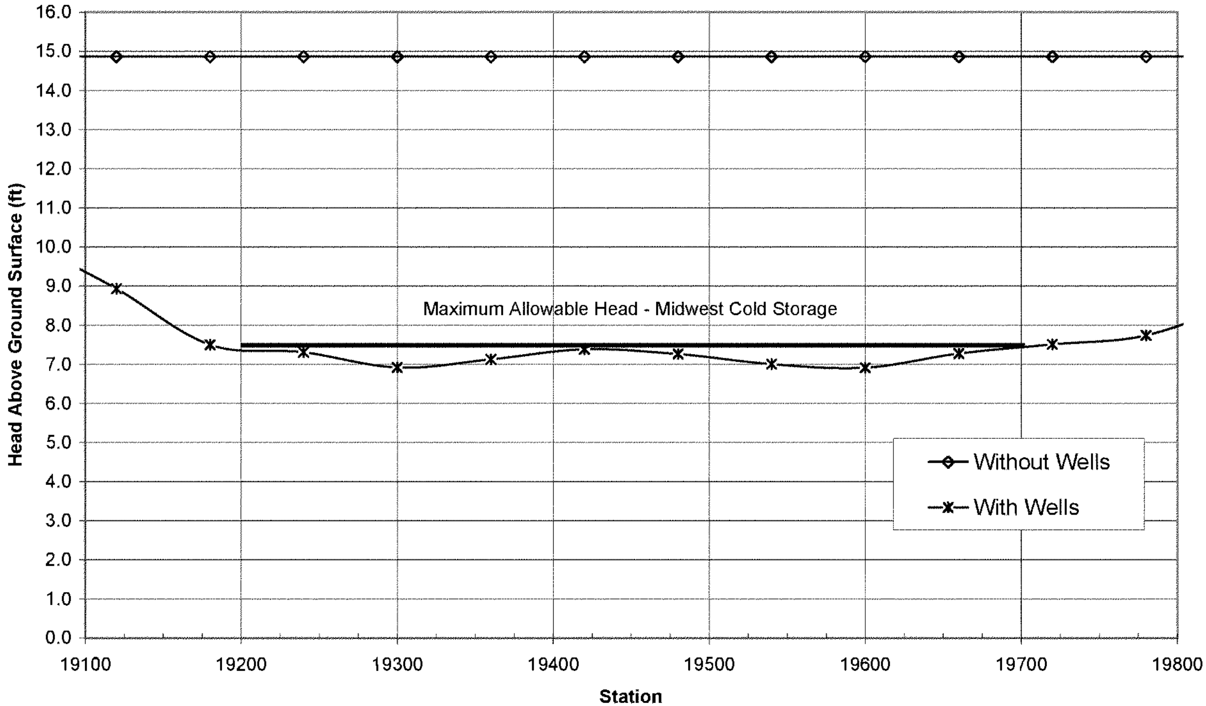


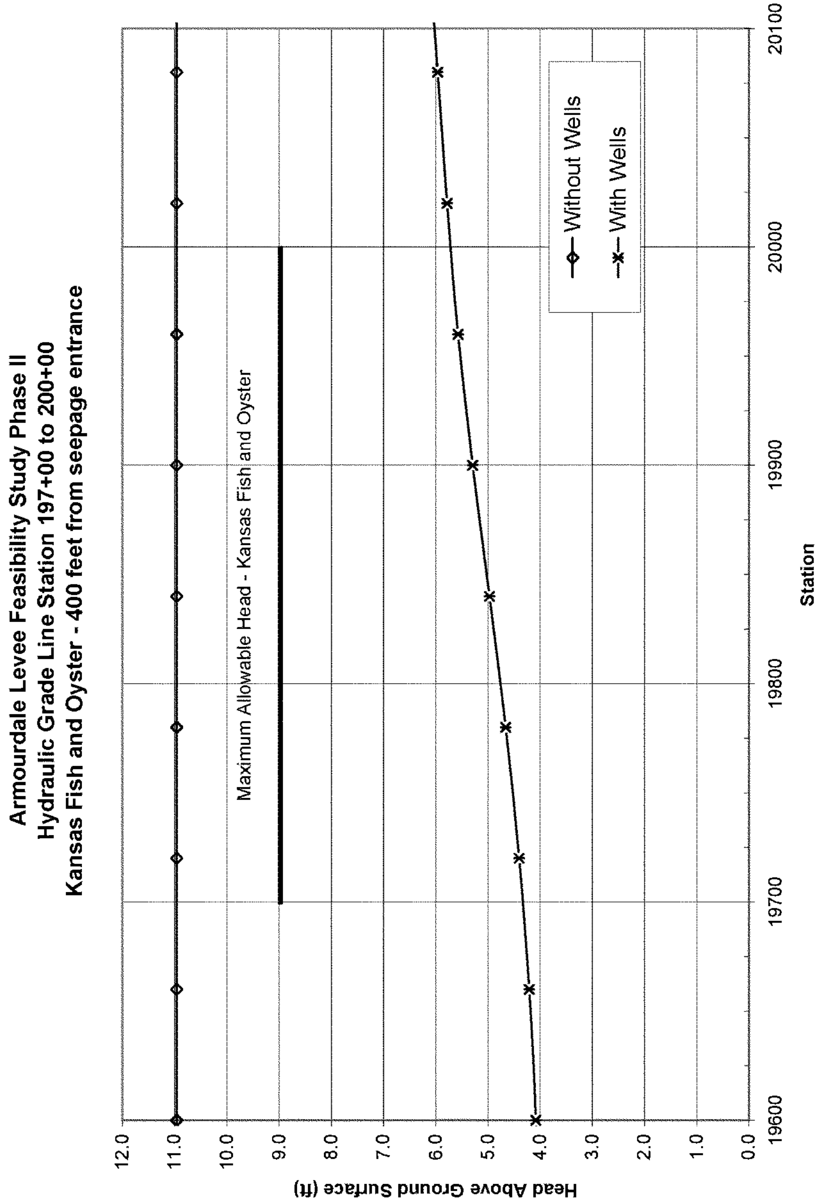
Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 251+00 to 254+00
Landside Toe
Head Above Ground Surface (ft)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
25100
25200
25300
25400
Station
Without Wells
With Wells
FSreq

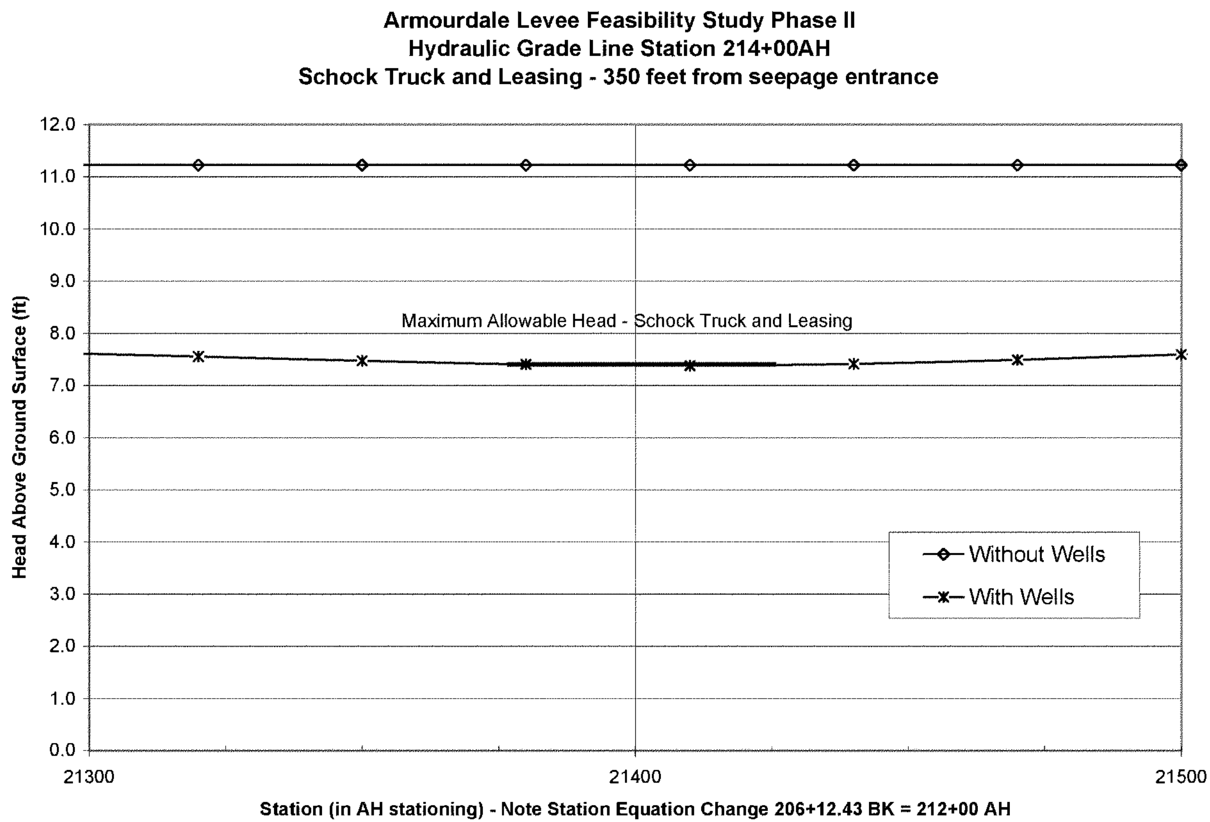
EXHIBIT A-3.20

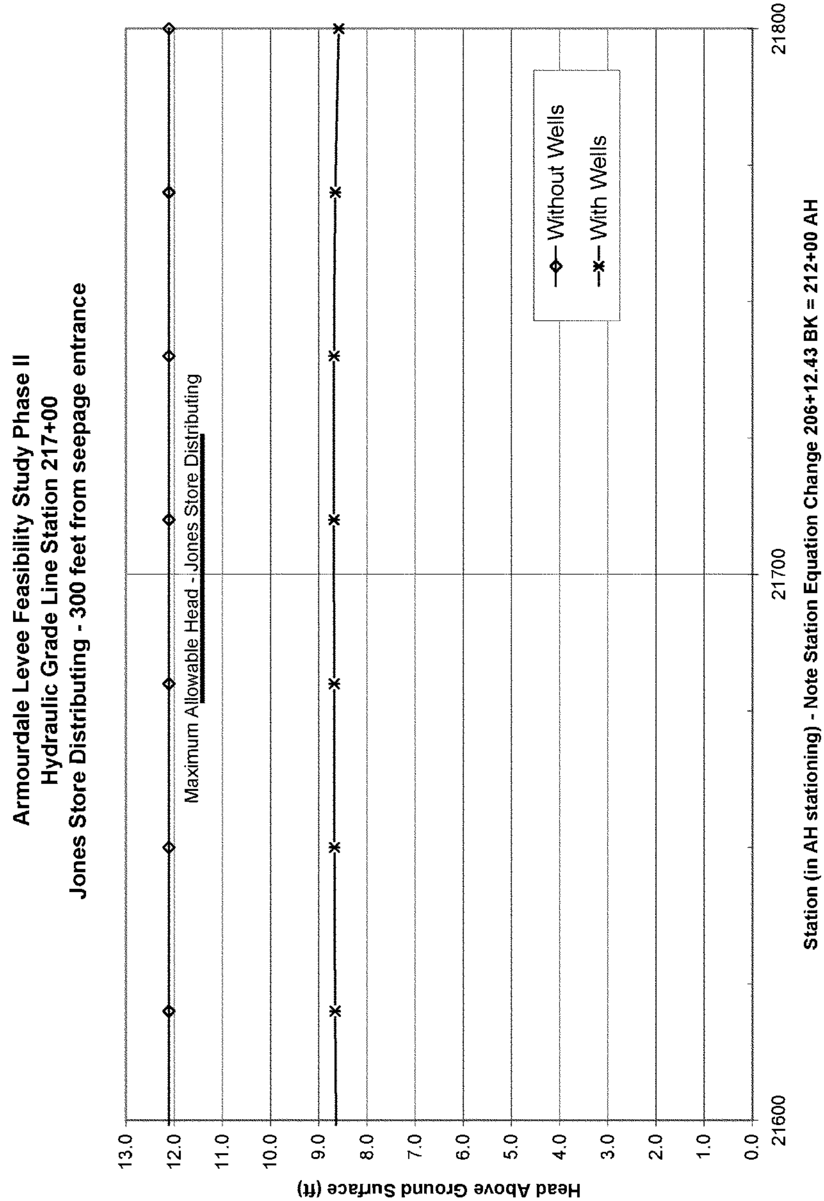
Computed Excess Head at Special Features

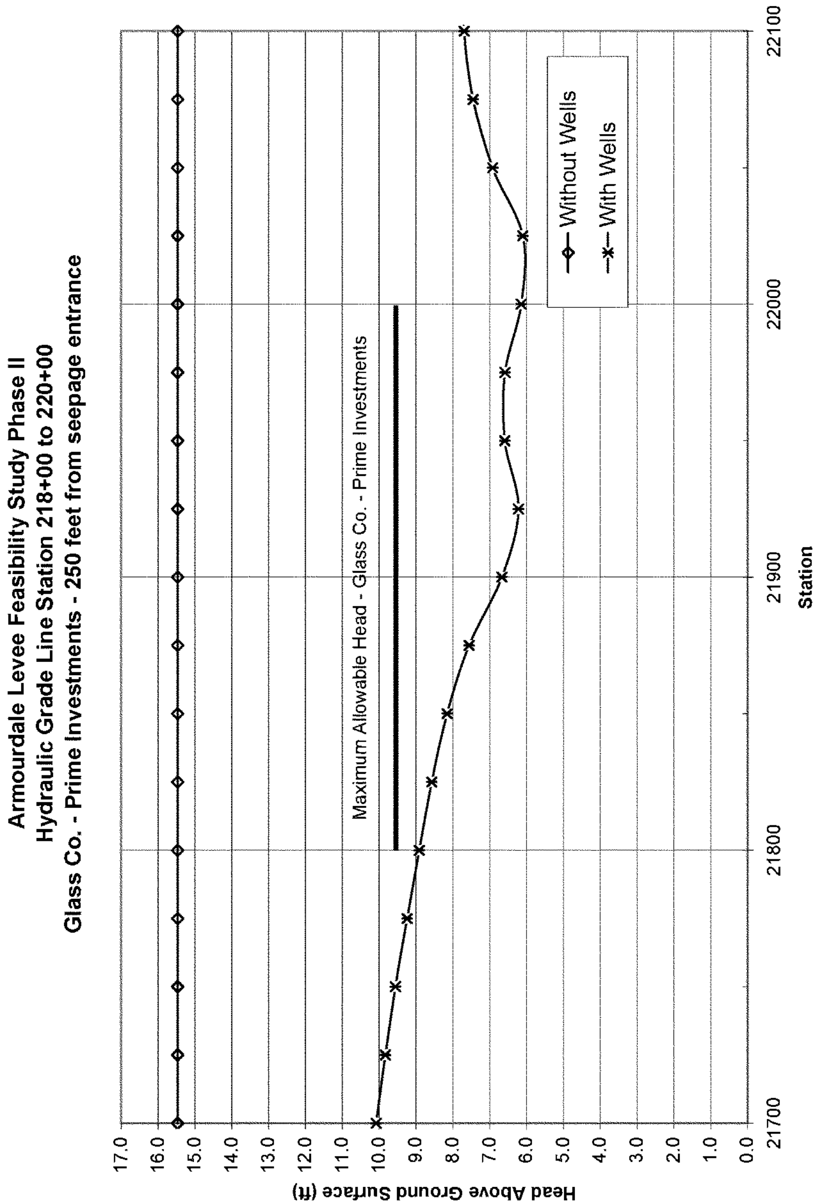
Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 192+00 to 197+00
Midwest Cold Storage - 200 feet from seepage entrance



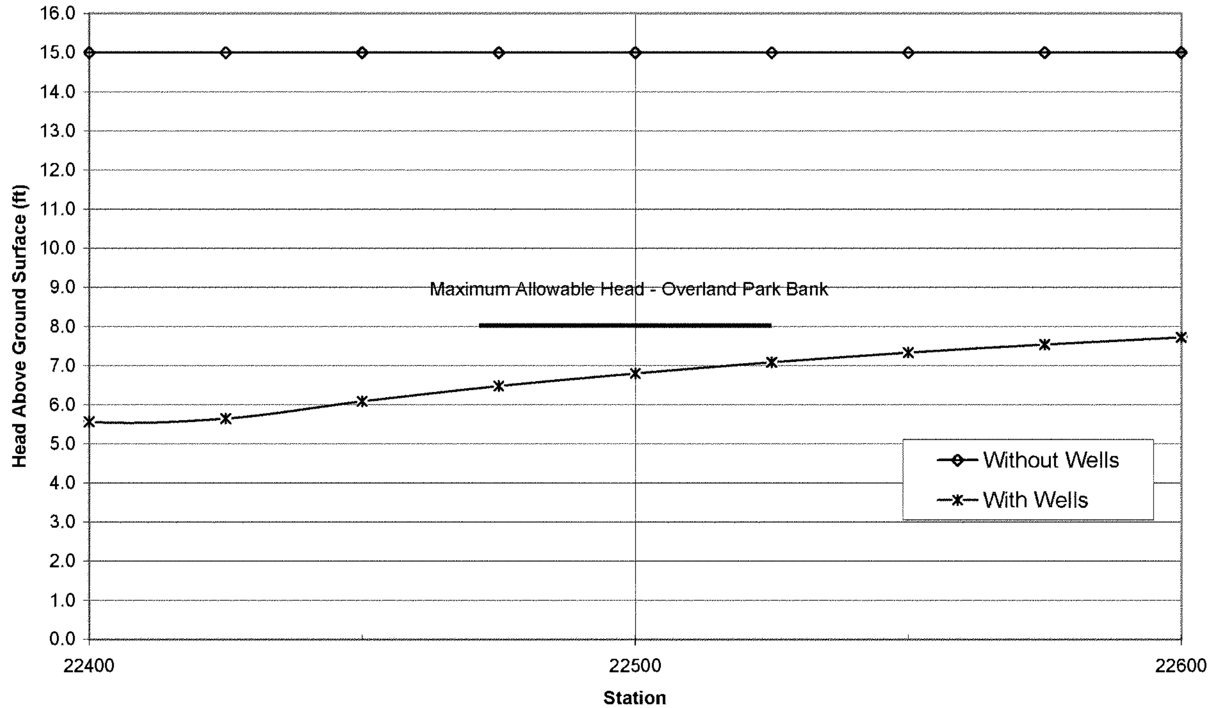

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 197+00 to 200+00
Kansas Fish and Oyster - 400 feet from seepage entrance
Head Above Ground Surface (ft)
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
19600
19700
19800
19900
20000
20100
Station
Maximum Allowable Head - Kansas Fish and Oyster
Without Wells
With Wells

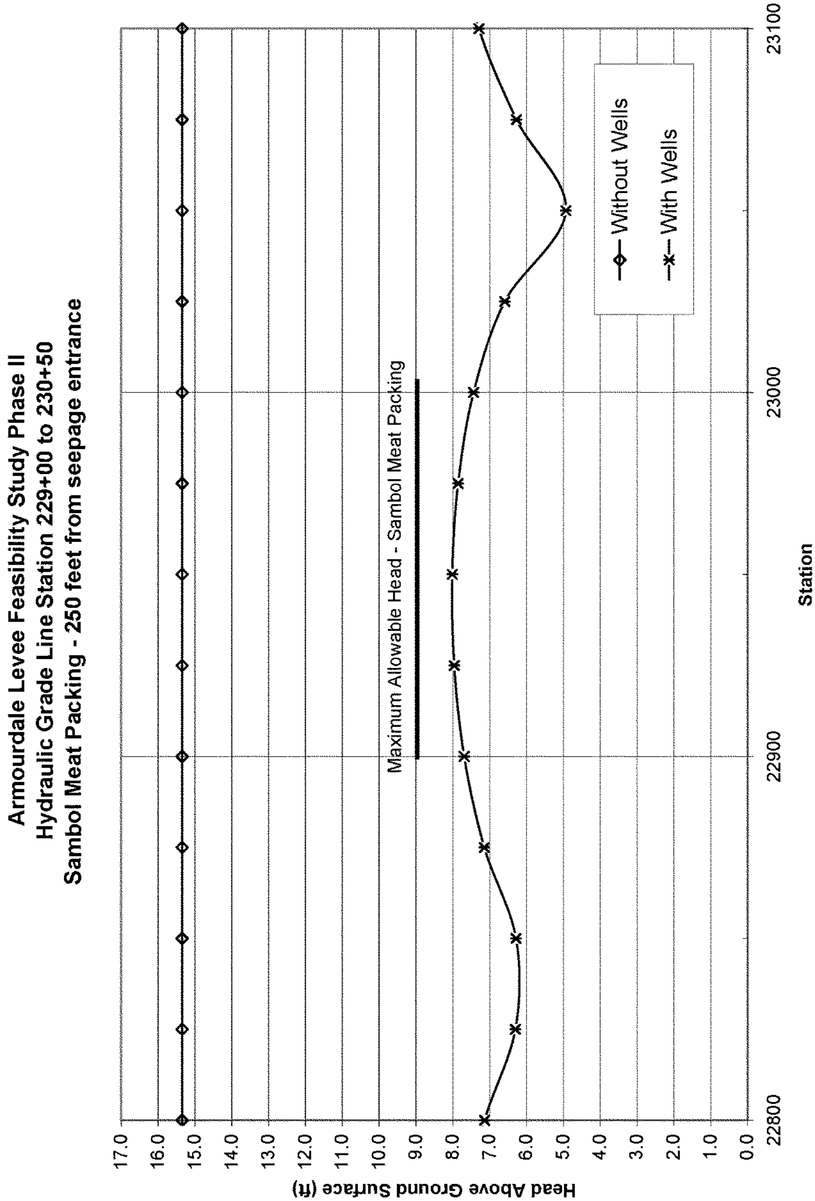

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 214+00AH
Schock Truck and Leasing - 350 feet from seepage entrance
Head Above Ground Surface (ft)
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Maximum Allowable Head - Schock Truck and Leasing
Without Wells
With Wells
21300
21400
21500
Station (in AH stationing) - Note Station Equation Change 206+12.43 BK = 212+00 AH

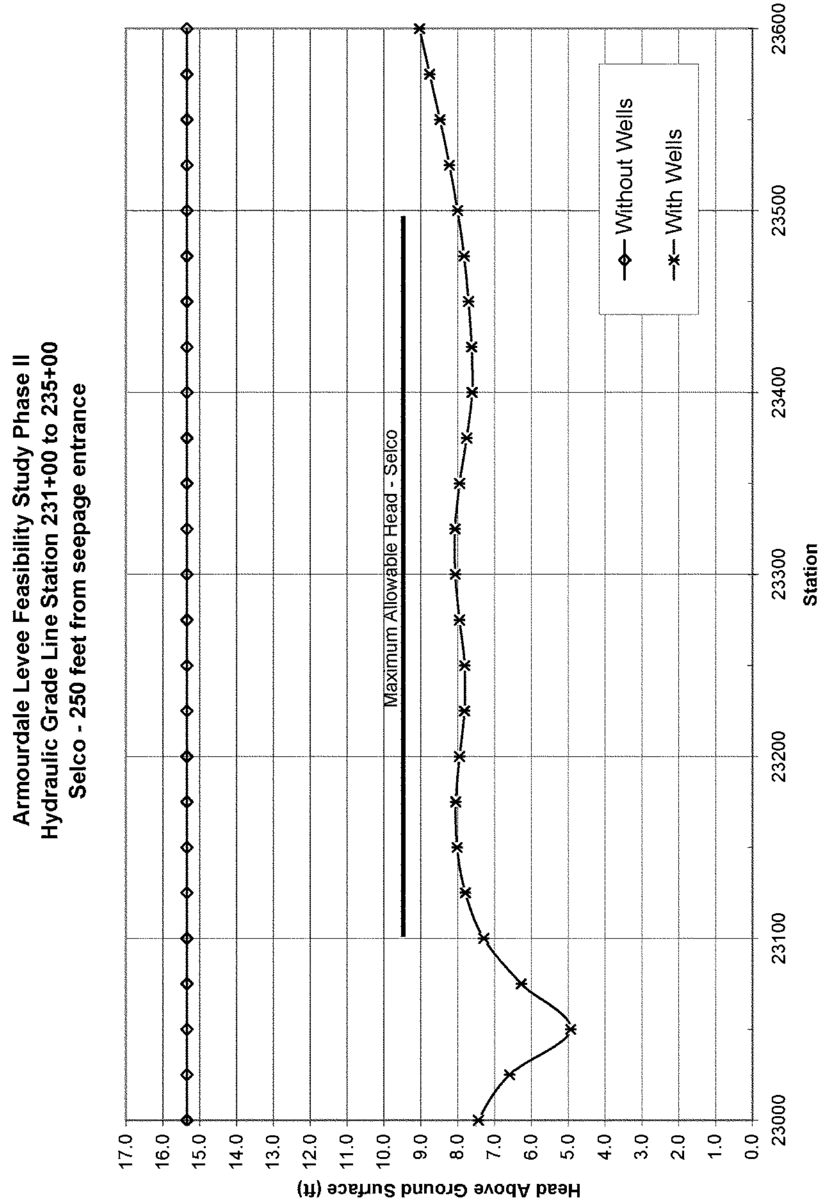

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 217+00
Jones Store Distributing - 300 feet from seepage entrance
Maximum Allowable Head - Jones Store Distributing
Without Wells
With Wells
Head Above Ground Surface (ft)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
21600
21700
21800
Station (in AH stationing) - Note Station Equation Change 206+12.43 BK = 212+00 AH


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 218+00 to 220+00
Glass Co. - Prime Investments - 250 feet from seepage entrance
Head Above Ground Surface (ft)
Station
Maximum Allowable Head - Glass Co. - Prime Investments
Without Wells
With Wells
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
21700
21800
21900
22000
22100

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 225+00
Overland Park Bank - 270 feet from seepage entrance




Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 229+00 to 230+50
Sambol Meat Packing - 250 feet from seepage entrance
Head Above Ground Surface (ft)
Station
Maximum Allowable Head - Sambol Meat Packing
Without Wells
With Wells
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
22800
22900
23000
23100


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 231+00 to 235+00
Selco - 250 feet from seepage entrance
Head Above Ground Surface (ft)
Station
Maximum Allowable Head - Selco
Without Wells
With Wells
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
23000
23100
23200
23300
23400
23500
23600

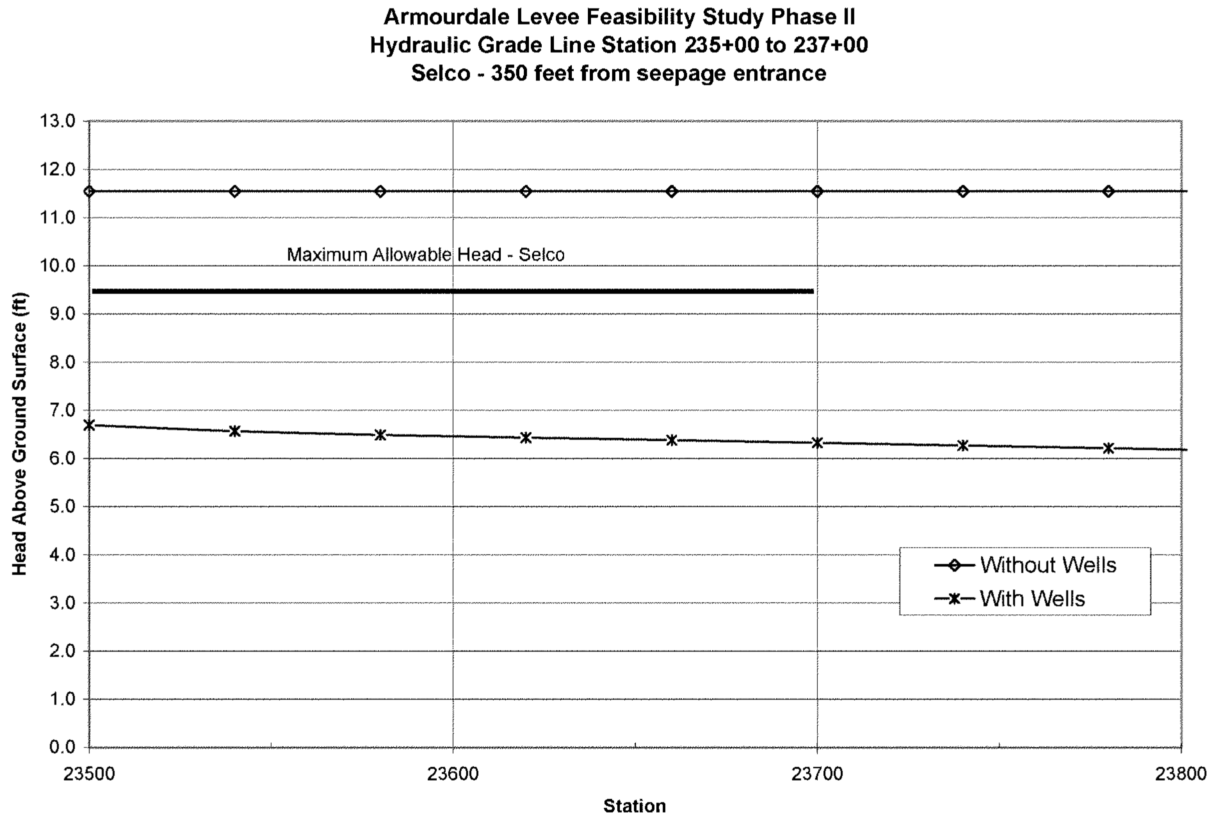

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 235+00 to 237+00
Selco - 350 feet from seepage entrance
Head Above Ground Surface (ft)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Maximum Allowable Head - Selco
Without Wells
With Wells
23500
23600
23700
23800
Station

Exhibit A-3.21

Proposed Relief Well System, Station 296+00 to 313+00

Status	Well	Distance From Seepage Entrance (ft)	Station (AH)	Discharge Elevation (ft)	Well Flow (cfs)
Proposed	1	220	29500	745	0.54
Proposed	2	220	29525	745	0.50
Proposed	3	220	29550	745	0.48
Proposed	4	220	29575	745	0.46
Proposed	5	220	29600	745	0.46
Proposed	6	220	29625	745	0.45
Proposed	7	220	29650	745	0.46
Proposed	8	220	29690	745	0.46
Proposed	9	220	29730	745	0.47
Proposed	10	220	29770	745	0.47
Proposed	11	220	29810	745	0.63
Proposed	12	220	29850	745	0.57
Proposed	13	220	29890	745	0.54
Proposed	14	220	29925	745	0.51
Proposed	15	220	29960	745	0.50
Proposed	16	220	29990	745	0.49
Proposed	17	220	30030	745	0.51
Proposed	18	220	30070	745	0.54
Proposed	19	220	30110	745	0.57
Proposed	20	220	30150	745	0.62
Proposed	21	220	30190	745	0.49
Proposed	22	220	30230	745	0.86
Proposed	23	220	30265	745	0.80
Proposed	24	220	30300	745	0.78
Proposed	25	220	30350	745	0.77
Proposed	26	220	30390	745	0.77
Proposed	27	200	30450	747	0.69
Proposed	28	200	30500	747	0.70
Proposed	29	200	30550	747	0.73
Proposed	30	200	30660	747	0.81
Proposed	31	200	30800	747	0.88
Proposed	32	200	30940	747	0.91
Proposed	33	200	31080	747	0.94
Proposed	34	200	31210	747	0.96
Proposed	35	200	31300	747	1.02

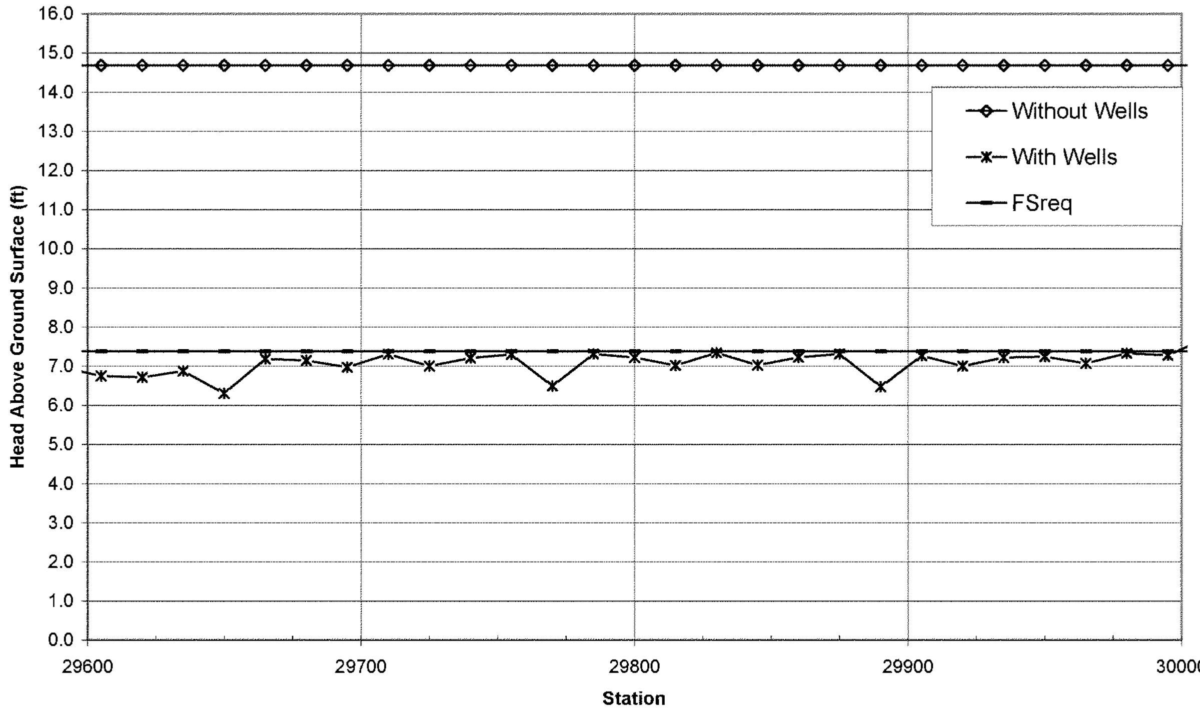
*Relief well flows shown are expected flows. The flows have been reduced by 80% in the calculations to account for future well efficiency reduction.

*Seepage entrance is assumed to be at the elevation of the bottom of the riverside blanket

EXHIBIT A-3.22

Computed Excess Head at Low Lying Area

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 296+00 to 300+00
Edge of Low Area ~220 feet from Seepage Entrance



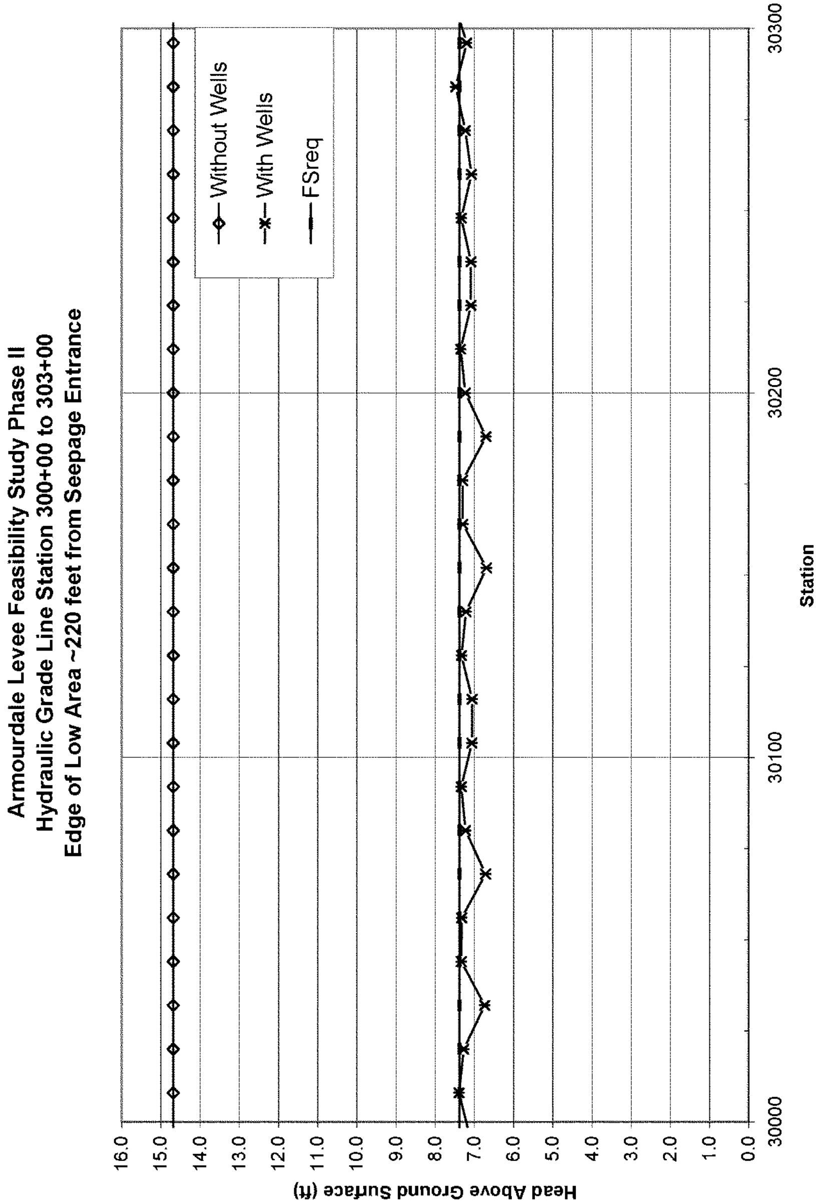
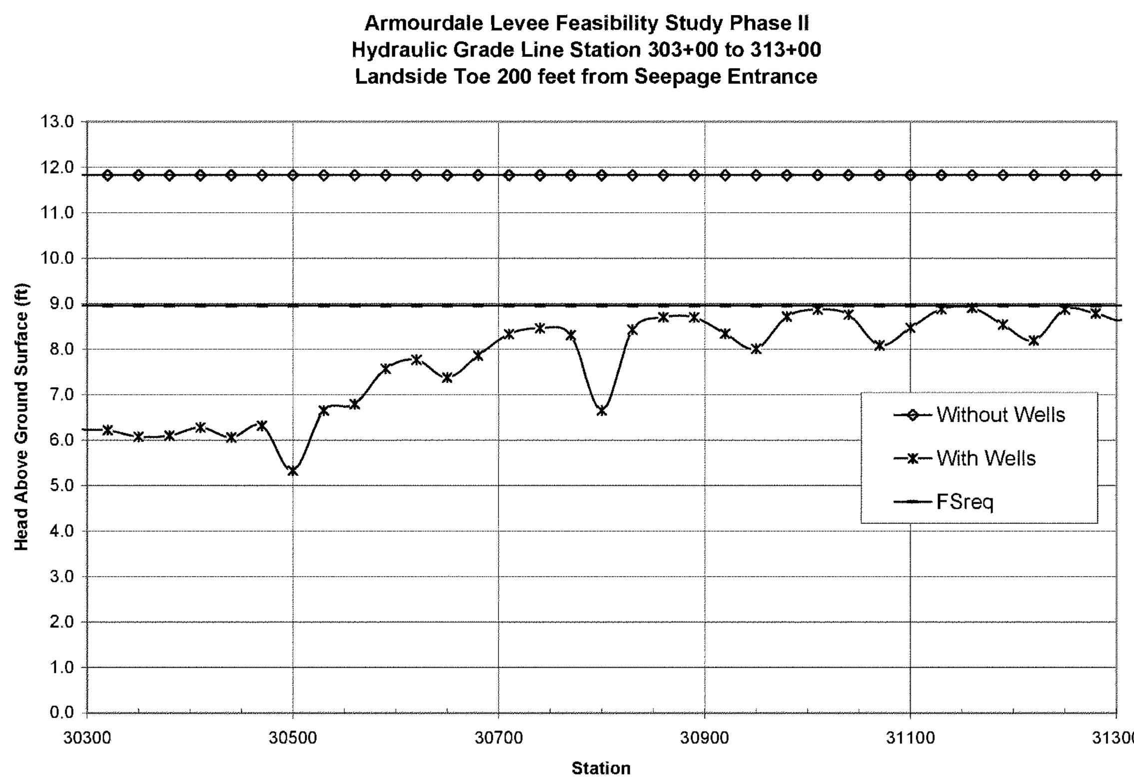

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 300+00 to 303+00
Edge of Low Area ~220 feet from Seepage Entrance
Head Above Ground Surface (ft)
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
30000
30100
30200
30300
Station
Without Wells
With Wells
FSreq

Exhibit A-3.23



Kansas Citys, Missouri and Kansas
Flood Risk Management Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Final Feasibility Report

Chapter A-4

GEOTECHNICAL ANALYSIS CID-KS

CHAPTER A-4
GEOTECHNICAL ANALYSIS – CID KS

A-4.1 INTRODUCTION

This chapter of the engineering appendix presents the results of the geotechnical evaluation performed for the Central Industrial District Levee Unit - Kansas (CID-KS) in the Kaw Valley Drainage District. The evaluation started with a thorough review of existing project documentation, definition of existing subsurface conditions along the entire unit based upon existing subsurface information, and estimation of soil parameters for the existing levees, the natural blanket and the aquifer materials. The estimated soil parameters are based on geotechnical laboratory testing data from Design Memorandum No. 4 – Central Industrial District Unit. All elevations used in the geotechnical portion of the feasibility study are NGVD 29.

Geotechnical analysis of the unit consisted mainly of underseepage and stability calculations for the following loading conditions:

- Existing Conditions, to identify the most critical areas with respect to risk of failure for use in the HEC-FDA economic model.
- Proposed Design Conditions, all of which included raising the current level of protection as follows:
    - Nominal 500 year flood event (N500+0), i.e. a 0.2% chance of occurrence in any one year
    - Nominal 500 year flood event plus 3-ft (N500+3)
    - Nominal 500 year flood event plus 5-ft (N500+5)

The majority of the design work focused upon the N500+3 flood event. The raise above the current level of protection varies from 0-ft to 3.8-ft, generally increasing in an upstream direction. For cost estimating purposes, the N500+0 and the N500+5 water elevations will be considered by interpolating and extrapolating the N500+3 results.

Underseepage was addressed along the entire CID-KS Unit. Where calculations showed hydraulic gradients in the natural blanket did not meet current criteria for the N500+3 design condition, seepage control measures were designed to reduce the gradient to meet criteria.

Slope stability analyses of the levee raises were not specifically performed for this unit. Since the foundation conditions and river loading were similar to the Armourdale Unit on the opposite side of the Kansas River, the results of the analyses from the Armourdale Unit were applied to the CID-KS Unit. The sections were conservatively compared to the Armourdale Unit based on the initial height of the protection, the amount of raise proposed, and the pore pressures in the natural blanket based upon the underseepage calculations. Additional stability studies should be performed during the design phase to verify these assumptions and possibly economize the proposed sections.

The results of the analyses are discussed in additional detail in later sections. For the purposes of economic modeling of proposed levee raises, all features which meet current Corps of Engineers criteria are arbitrarily assigned a reliability of 99.8%.

A-4.2 DESCRIPTION OF EXISTING LEVEE UNIT

A-4.2.1 Levee Description

The CID-KS Unit is located in Wyandotte County, Kansas and extends along the right bank of the Kansas River from mile 3.4 to the mouth, then downstream along the right bank of the Missouri River to the Missouri and Kansas State line where the CID-MO Unit begins (the two units are directly connected and there is no hydraulic separation). The levee unit begins at the Kansas and Missouri State line at Station 83+01.29 CID-MO and extends to Station 89+37.34 CID-MO where the stationing changes to Station 0+00 CID-KS. The levee extends to Station 168+49 CID-KS where the unit ties into high ground just downstream of where the mouth of the Turkey Creek Tunnel discharges into the Kansas River. The unit consists of a system of levees and floodwalls, stoplog gaps, sandbag gaps, pumping plants, riprap and levee toe protection, and surfaced levee crown and ramps. The greater portion of the area to be protected consists of 360 highly industrialized areas. These areas are occupied largely by railroads, wholesale houses, and manufacturing plants. The area of interior drainage also includes 352 acres along the bluffs to the south and east. The total length of the unit is 17,485 feet or 3.3 miles.

There are many bridges, structures, and utilities within the critical area of the line of protection. For the purposes of the Geotechnical Analysis, it was assumed that all bridge foundation elements, structures, and utilities within the levee embankment and critical area of the foundation blanket material meet all pertinent Corps of Engineers criteria. Henceforth, no analysis was completed regarding their integrity or changes to due to the proposed levee raise.

A-4.2.2 History

The Kaw Valley Drainage District initially began work on the CID-KS Unit prior to any involvement by the Federal Government. Previous works included the construction of earthen levee sections, drainage structures and floodwalls. The floodwalls were constructed by the Works Projects Authority (W.P.A.) in the mid 1930s. The Flood Control Act of 1936 authorized the Corps of Engineers to provide assistance. Work began to improve parts of the project in the late 1940s. The flood of 1951 caused extensive damage to the original levee, and the Corps of Engineers designed and constructed the restoration of the protection. The Corps of Engineers again became involved in the 1960s to raise the level of protection along the CID-KS Unit, reference Design Memorandum No. 4. The raise was constructed in the late 1970s. The following discussion describes the existing unit in additional detail by major features.

<u>Station 83+01.29 CID-MO to 25+90 CID-KS</u>

This is a levee section that starts at the Missouri and Kansas State line and extends to the James Street Bridge. This section was constructed on a large pervious fill. There is a buried collector system that extends from across the state line and terminates at Station 5+00 CID-KS to collect underseepage through the fill.

Station 25+90 to 26+72.66
This section is a sand bag gap across the James Street Bridge.

Station 26+72.66 to 40+31.25
This section is a timber pile founded floodwall that extends from the James Street Bridge to the Kansas City Southern Railroad Bridge Abutment.

Station 40+31.25 to 41+00
This section consists of the Kansas City Southern Railroad Bridge Fill.

Station 41+00 to 74+35.94
This is a levee section which extends from the Kansas City Southern Railroad Bridge fill to near the Stockyards No. 3 Pump Station. The original section had an internal embankment pervious section with a lateral drain pipe to improve the internal embankment through seepage condition. It is unclear if this drain pipe was plugged during the 1962 Modification levee raise. If this section is to remain in place, it should be evaluated for stability during PED.

Station 74+35.94 to 77+27.75
This section consists of tie back concrete pile founded flood walls, the Missouri Pacific Railroad Bridge fill, and a sand bag gap across the Union Pacific Railroad Bridge.

Station 77+27.75 to 102+73.38
This is a small levee section built on a large fill which is supported by a large landward retaining wall. There is a series of 10 relief wells at the landward toe of the retaining wall with a header system that transports the relief well flows to the Stockyard No 3 Pump Station. There is little information known about the backfill or construction of the landward retaining wall. While failure of the wall will not directly affect the levee section, it may impede the effectiveness of the relief wells. If this wall is to remain in place it should be evaluated for global and local stability during PED.

Station 102+73.38 to 168+49
This section is a timber pile founded floodwall section. This reach consists of several gap closure structures. The former stoplog gap at the CRI&P Railroad bridge crossing at Station 104+51 has been permanently closed. The stoplog gaps at the KC Terminal Railroad Bridge at Station 132+20 and at the upper end of the unit across several sets of tracks at Station 168+00 are still active.

Based upon the record drawings, the existing levee sections have a thick impervious core with a riverside pervious section protected by riprap and a random or pervious landside section.

A-4.2.3 General Geology of the Region (Kansas River)

The Kansas River Valley, near its mouth, is cut into Pennsylvanian bedrock of the Missourian Series. The oldest bedrock exposed is the Bethany Falls Limestone member of the Swope Limestone formation, Kansas City Group. Bedrock of the Missourian Series is characterized by numerous limestone beds separated by clayey to somewhat sandy shale. The bedrock is generally overlain by much younger unconsolidated materials consisting of glacial drift, loess of the Pleistocene age, alluvium deposits and isolated remnants of till of Kansas stage ice sheet occurring on the hilltops. The Kansas River is near the southern edge of Kansas glaciation. Wind-blown deposits of silt (loess) form an irregular deposit covering much of the eastern part of Wyandotte County. Alluvium, ranging from clay and silt to sand and gravel, occurs in the Kansas River Valley. Much of this alluvium is probably of glacial origin, having been deposited as glacial outwash from the melting ice sheets.

A-4.2.4 Subsurface Conditions

Assessments of the subsurface conditions for the CID-KS project were derived from the Record Drawings, Design Memorandums and borings made at selected sites during Phase 1 of the feasibility study. Typical subsurface conditions for the CID-KS Unit are zero to 15-ft of fill overlying 20-ft to 30-ft of slits and clays overlying 50-ft to 70-ft of sand overlying bedrock. The composition of the fill is highly variable and consists of earth, organics, cinders, bricks, and other construction debris. The fill was considered to be a part of the blanket unless boring logs indicated it was made up of more than 70% pervious material or if the boring logs did not clarify the composition of the fill. Groundwater levels are dependent on the seasonal changes and rises in the river. The water levels observed during drilling are shown on drill logs and recorded on the strip log summary of the Record Drawings and Design Memorandum No. 4. In general the water levels measured adjacent to the existing level of protection were on average approximately 20-ft below the landside ground surface for normal river levels.

A-4.2.5 Existing Underseepage Control Features

Throughout the existence of the CID-KS Unit, many underseepage control measures have been constructed to aid in the prevention of developing an underseepage condition that could cause a levee failure. Underseepage control measures were designed and constructed during original construction and during the 1962 Modification of the unit.

The existing underseepage control features are detailed below. They were generally designed and constructed as a part of the 1962 Modification. Additional details can be found in Design Memorandum No. 4.

Station 82+61 CID-MO to 5+00 CID-KS

A buried collector system was constructed in this reach to collect seepage which flows through a large sand fill underneath the flood protection. Details on the collector system can be found in the CID-MO Unit Restoration Drawings Volume 1, Sheet No. 17-21 and CID-KS Record Drawings Volume 2, Sheet No. 20.

<u>Station 63+60 to 81+50</u>
An area fill was constructed in a low lying area landward of the levee. The area fill was constructed to elevation 747 and extends up to approximately 200-ft landward of the levee centerline. Details on the area fill can be found in the CID-KS Unit Record Drawings Volume 2, Drawing No. A6-A7. This area fill replaced a series of relief wells.

<u>Station 79+00 to 97+00</u>
A relief well system, consisting of 10 fully penetrating artesian relief wells, is in place to remediate an underseepage concern in the Stockyards Area. The relief well system was designed to provide a factor of safety with respect to hydraulic gradient of 1.5 at all check points, and 1.0 in basements, pits, or low spots with the water at the top of the levee. It is unclear if the entire Stockyard area was treated as a low spot similar to the "slot area" across the river on the Armourdale Unit. The wells are variably spaced and connected by a gravity header system which discharges into the Stockyards No 3 Pump Station. The flow from each well was assumed to be 1.0 cfs in the original design. Details on the relief well system can be found in the CID-KS Unit Record Drawings Volume 2, Drawing No. A7-A8 and C1-C3.

A-4.2.6 Overall Underseepage

For the underseepage analysis, the entire CID-KS Unit was divided into reaches of similar protection height, blanket thickness, blanket composition, aquifer thickness, and seepage entrance conditions. The factor of safety with respect to hydraulic gradient through the natural blanket was calculated for each of these reaches at the landside toe of the levee section or floodwall. Exhibit A-4.1 shows the calculated factor of safety with respect to hydraulic gradient for the entire CID-KS Levee Unit (without the effects of existing relief wells under existing conditions), as well as the parameters used to calculate the factor of safety with respect to hydraulic gradient. Exhibit A-4.1 also includes a subsurface profile along the entire levee centerline and is included in the supplemental exhibits section. Parameters were obtained from the Record Drawings, Design Memorandum No. 4, and other historical design documents.

A-4.3 SOIL STRENGTH PARAMETERS

The required parameters for soils in the CID-KS Unit were estimated mainly from the significant amount of geotechnical laboratory testing performed for the 1962 Modification and provided in Design Memorandum No. 4. A summary of the soil parameters is provided in Table A-4.1 and is discussed in the following paragraphs.

The existing levee sections consist of a riverward impervious zone and landward random fill zone, and toward the lower end of the unit there is also a pervious fill section on the riverside. To simplify the analyses, one set of parameters was used for the entire levee section.

The blanket materials consist mostly of ML and CL materials, with some discontinuous layers of CH and SM material. The upper end of the project, however, has a significant zone of CH materials. Design Memorandum No. 4 presented the test results sorted by soil classification. To simplify the analysis for this study, the blanket was modeled as a single material with only one set of strength parameters used. The soil strength applied to the blanket was a weighted average of the strength parameters for CL, ML and CH from the Design Memorandum test data.

TABLE A-4.1
Geotechnical Design Parameters

Material	Unit Weight		Shear Strength			
	Moist	Saturated	Undrained		Drained	
	γ (pcf)	γ (pcf)	c (psf)	ϕ (deg)	c' (psf)	ϕ' (deg)
Levee Fill	115	120	1000	0	0	29
Foundation Blanket	110	115	500	0	0	24
Foundation Sands	115	120	N/A	N/A	0	31

The shear strength for the foundation sands was estimated from standard penetration test data performed in October 2001.

Undrained shear strength data was not readily available for most of the materials, so undrained strengths were estimated from limited 1962 Modification test data for CH soils and typical values for the materials. The undrained strength data for the Armourdale Unit was also considered. Foundation blanket strength data was reduced from the existing CH test data for CID-KS due to the limited amount of data. It is recommended that additional sampling and testing be performed during PED to verify the strengths of the blanket, levee, and aquifer materials.

A-4.4 EXISTING CONDITIONS RELIABILITY ANALYSIS

A-4.4.1 Introduction

The purpose of this portion of the study was to determine the probability of failure of the CID-KS Levee Unit for the existing condition of the unit. The analysis considered both underseepage piping failures and landward slope failures under steady state seepage conditions. The evaluations were performed in general accordance with the USACE Engineering Technical Letter (ETL) 1110-2-556 "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies." The results of the analyses were used to determine the economic benefits attributed to proposed levee raises.

A-4.4.2 Probabilistic Theory

A-4.4.2.1 Probabilistic Parameters

Several parameters are commonly used to describe probability distributions such as the normal distribution shown in Exhibit A-4.2 at the end of this chapter. Probably the most common of these is the mean or expected value. The expected value of a continuous random variable X (a variable that can take on any value within some continuous range) with some distribution $f(x)$ is defined as:

$$\mu_X = \int_{-\infty}^{\infty} x_i f_X(x)\ dx \qquad \text{Equation A-4.1}$$

where μ_X is the mean value of the random variable X, x_i is a particular value of the random variable X and $f_X(x)$ is the frequency of occurrence of the random variable X. The expected value, or mean, of a random variable is the weighted average of the values of the random variable with the weighting being the frequency of occurrence of the value. For a set of discrete measurements of a random variable, the mean value is computed as:

$$\mu_X = \frac{\sum_{i=1}^{N} x_i}{N} \qquad \text{Equation A-4.2}$$

The variance of the random variable X, Var[X], is a measure of the spread, or variability of the random variable about the mean. The variance is computed as:

$$\text{Var[X]} = \int_{-\infty}^{\infty} (x_i - \mu_X)^2 f_X(x)\ dx \qquad \text{Equation A-4.3}$$

For a set of discrete measurements of a random variable X, the variance is computed as:

$$\text{Var}[X] = \frac{\sum_{i=1}^{N} (x_i - \mu_X)^2}{N} \qquad \text{Equation A-4.4}$$

If the number of observations N is a relatively small set of an entire population, an unbiased estimate of the variance can be given as:

$$\text{Var}[X] = \sigma_X^2 = \frac{\sum_{i=1}^{N} (x_i - \mu_X)^2}{N-1} \qquad \text{Equation A-4.5}$$

The standard deviation, σ_X, is also a measure of the distribution of the random variable about the expected value and is the square root of the variance:

$$\sigma_X = \sqrt{Var[X]} \qquad \text{Equation A-4.6}$$

The coefficient of variation, COV, is a convenient dimensionless parameter used to express the uncertainty or variability of a random variable and is computed as:

$$\mathrm{COV} = \frac{\sigma_X}{\mu_X} \qquad \text{Equation A-4.7}$$

The coefficient of variation is useful because it expresses the variability of a random variable normalized with respect to the mean of the random variable. The expected value, standard deviation and coefficient of variation are interrelated; therefore, the third can be determined by knowing any two of the parameters.

A-4.4.2.2 Probability Distributions

Many forms of probability distribution are available that can be used to represent the variability and uncertainty. However, based on previous work (Kitch, 1994) the normal and log-normal distributions are by far the most commonly used for risk based analyses.

The normal distribution is the most widely used distribution in the description of statistical phenomenon. The probability density function for a normally distributed random variable is expressed as:

$$f_X(x) = \frac{1}{\sigma_X\sqrt{2\pi}}\exp\left[-\frac{1}{2}\left(\frac{x-\mu_X}{\sigma_X}\right)^2\right]dx \qquad \text{Equation A-4.8}$$

where $f_X(x)$ is the relative frequency of the random variable X and is not a probability, but a representation of the distribution of probability that a particular random variable may lie within some stated interval. As shown in Exhibit A-4.2 the normal distribution has a bell shape with upper and lower limits of positive and negative infinity.

Another distribution that has been proven useful for reliability-based analysis in geotechnical engineering is the log-normal distribution shown in Exhibit A-4.3 at the end of this chapter. In the log-normal distribution, it is assumed that the natural logarithm of a random variable X is normally distributed. As shown in Exhibit A-4.3, the log-normal distribution is positively skewed towards the lower values. However, it has the distinct advantage that the probability of the random variable cannot be less than zero. The log-normal distribution is therefore useful for representing parameters that cannot take on negative values (e.g. factors of safety and hydraulic gradient).

If a random variable X is log-normally distributed, the *ln X* is normally distributed. The probability density function can therefore be expressed as:

$$f_X(x) = \frac{1}{x\sigma_{\ln X}\sqrt{2\pi}}\exp\left[-\frac{1}{2}\left(\frac{\ln x - E[\ln X]}{\sigma_{\ln X}}\right)^2\right]dx \qquad \text{Equation A-4.9}$$

where $\sigma_{\ln X} = \sqrt{Var[\ln X]}$, and $E[\ln X]$ is the expected value(mean) of the natural logarithm of X.

A-4.4.2.3 Probabilistic Measure of Slope Stability

In reliability-based analysis of slope stability, the input parameters that are not well defined are considered to vary according to some form of distribution as described in the previous section. These variable parameters are then used as input into a series of stability analyses to obtain the overall distribution of the performance function. The performance function is used to report the stability of the slope. The performance function used throughout this study for slope stability is the factor of safety.

A hypothetical distribution of the factor of safety that could result from analyses using probabilistic parameters is shown in Exhibit A-4.4 at the end of this chapter. As shown in the figure, the distribution indicates that the actual factor of safety may take on a range of possible values, ranging from well below the limiting value of FS = 1.0 to well above the limiting value. While knowledge of the complete distribution of the factor of safety is useful, it is the relative frequency of factors of safety less than the limiting value that are of primary importance (FS ≤ 1.0 => Failure). Three different probabilistic parameters are typically used to represent this relative frequency.

The probability of failure of a system is the area under the probability density function shown as the shaded area in Exhibit A-4.4. For the log-normal function, this would be from the boundaries (0 ≤ FS ≤ 1). In mathematical terms it can be expressed as:

$$P_f = \int_0^1 f_X(x)\,dx \qquad \text{Equation A-4.10}$$

where $f_X(x)$ is the probability density function expressed in Equation A-4.8.

The reliability of a system is conversely the area under the probability density function bounded by the limiting value and positive infinity. In Exhibit A-4.4, it is represented by the non-shaded area under the curve. For a log-normal distribution, the boundaries would be (1< FS ≤ +∞). Since the total probability for all possible values of the random variable is 1.0, the probability of failure, P_f, and the reliability, denoted as R, are related by:

$$P_f = 1\text{-}R \qquad \text{Equation A-4.11}$$

Based on the assumption that the factor of safety is log-normally distributed, the natural log of the factor of safety will be normally distributed. In this case, the boundaries for the probability of failure would be (-∞< lnFS ≤ 0). Under this assumption, the probability curve and its probabilistic parameters would be represented in Exhibit A-4.5, in the supplemental exhibits section, with the probability of failure in the shaded area.

The reliability index, β , is a gage of the reliability of a system that takes into account technicalities of the procedure and the uncertainties introduced by random input variables. The reliability index gives a measure of comparative reliability for a system, thereby making it unnecessary to calculate or determine the actual probability distribution. It is defined using the probabilistic terms of standard deviation and the expected value (mean) of the performance function. Graphically, the reliability index multiplied by the standard deviation is equal to the distance from the expected value (mean) to the limiting state as shown in Exhibit A-4.4. For a log-normal distribution, the reliability index is computed as:

$$\beta = \frac{\ln\left[E[FS] \Big/ \sqrt{1+COV[FS]^2} \right]}{\sqrt{\ln\left(1+COV[FS]^2\right)}} \qquad \text{Equation A-4.12}$$

where β is the reliability index, *E[FS]* is the expected value (mean) of the factor of safety, and *COV[FS]* is the coefficient of variation of the factor of safety.

A-4.4.2.4 Probabilistic Measure of Stability for Underseepage

When the excess head at the ground surface on the landward side of the levee toe is greater than zero and the blanket material is thicker than one-fourth the levee height, the probability of failure can be calculated using the method described in ETL 1110-2-556.

Using this method, the exit gradient (i) is assumed to be a log-normally distributed random variable with probabilistic moments E[i] and σ_i. Based on this assumption, the equivalent normally distributed random variable has moments E[ln i] and $\sigma_{\ln i}$. The limit state for the underseepage would then be the natural log of the failure gradient (i_f) with the boundaries for the probability of failure being:

$$P_f = P(\ln i > \ln i_f) \qquad \text{Equation A-4.13}$$

The probability of the ln i being greater than the ln i_f is determined by using the standard normalized variate (z), which is also analogous to the reliability index β. The standard normalized variate is calculated as:

$$z = \beta = \frac{\ln i_f - E[\ln i]}{\sigma_{\ln i}} = \frac{\ln\left[\dfrac{i_f * \sqrt{1+COV[i]^2}}{E[i]} \right]}{\sqrt{\ln(1+COV[i]^2)}} \qquad \text{Equation A-4.14}$$

where, *E[i]* is the expected value (mean) of the hydraulic gradient and *COV [i]* is the coefficient of variation of the hydraulic gradient. Exhibit A-4.6, found in the supplemental exhibits section, shows a graphical representation of the probabilistic parameters for the underseepage analysis with the probability of failure in the shaded area.

A-4.4.2.5 Taylor Series Approximation Method for Determining Risk and Uncertainty Analysis

As described in the previous sections, the probability of failure can be computed if the expected value (mean) and variance of the distribution are known. Numerous methods are available for computing the probability of failure for reliability-based analyses, including first order second moment methods (FOSM), the point estimate method, the Hasofer-Lind method, and Monte Carlo simulations (Baecher & Christian 2000). While all of these methods can be used, the most commonly used method to date in geotechnical applications is the Taylor Series Approximation of the FOSM method (USACE, 1999). The basis of the Taylor series method is that it uses the first two linear terms on the Taylor series expansion of the performance function to determine the probabilistic measures of performance. As such, the method is exact for linear performance functions and is approximated for higher order functions. While this method is approximate from a strictly probabilistic point of view, it has the significant advantage of being relatively simple to implement.

For a function (Y) of random independent variables (X_1, X_2, . . .X_n) of the form

$$Y = g(X_1, X_2, \ldots X_n) \qquad \text{Equation A-4.15}$$

the expected value (mean) of Y can be found by evaluating the function at the expected values (mean) of the random variables. In the slope stability analysis application, the function Y is chosen to be the factor of safety and the random variables are the input parameters that are chosen as probabilistic. The expected value of the factor of safety is therefore computed directly from the expected values (mean) of the random variables.

Stated in mathematical form, this is:

$$E[FS] = FS(E[\overline{\phi}_{\text{foundation}}], E[\overline{\phi}_{\text{blanket}}], E[\overline{\phi}_{\text{embankment}}]) \qquad \text{Equation A-4.16}$$

where E[FS] is the expected value (mean) of the factor of safety and $E[\overline{\phi}_{\text{foundation}}]$, $E[\overline{\phi}_{\text{blanket}}]$, and $E[\overline{\phi}_{\text{embankment}}]$ are the expected values (mean) of the random variables.

The Taylor Series approximation for the variance of the factor of safety can be expressed as:

$$Var[FS] = \sum\left[\left(\frac{\partial FS}{\partial X_i}\right)^2 Var[X_i]\right] \qquad \text{Equation A-4.17}$$

where X_i represents a value of the ith random variable for the stability analysis, $Var[X_i]$ is the variance of that random variable, and $\frac{\partial FS}{\partial X_i}$ is the partial derivative of the distribution of the factor of safety evaluated at the expansion point. Noting that the $Var[X] = \sigma_X^2$ and approximating the partial derivative with a difference form, Equation A-4.17 becomes:

$$\text{Var[FS]} = = \sum\left(\left[\frac{\Delta FS}{\Delta X_i}\right]^2 \sigma_i^{\ 2}\right) \qquad \text{Equation A-4.18}$$

where σ_i is the standard deviation of the ith random variable and $\frac{\Delta FS}{\Delta X_i}$ is the approximated partial derivative. It has become common to evaluate the partial derivative $\frac{\Delta FS}{\Delta X_i}$ at the expected value (mean) plus one standard deviation and at the expected value (mean) minus one standard deviation as shown in Exhibit A-4.7, found at the end of this chapter, so that $\Delta X_i = 2\sigma_i$. Making this simplification, the expression for the variance becomes:

$$\text{Var[FS]} = \sum\left(\frac{FS(E[FS]+\sigma_{FS}) - FS(E[FS]-\sigma_{FS})}{2}\right)^2 \qquad \text{Equation A-4.19}$$

where $FS(E[FS] + \sigma_{FS})$ is the factor of safety calculated at the expected value plus one standard deviation and $FS(E[FS] - \sigma_{FS})$ is the factor of safety calculated at the expected value minus one standard deviation. Noting that the $\sqrt{Var} = \sigma$, the equation for the standard deviation for the factor of safety will become:

$$\sigma_{FS} = \sqrt{\left(\frac{\Delta FS_1}{2}\right)^2 + \left(\frac{\Delta FS_2}{2}\right)^2 + \ldots\ldots + \left(\frac{\Delta FS_n}{2}\right)^2} \qquad \text{Equation A-4.20}$$

where σ_{FS} is the standard deviation of the factor of safety and ΔFS is the difference between the factors of safety calculated at the expected value plus and minus one standard deviation for each of the random variables.

The discussion above describes how the factor of safety was evaluated as the limit state function. The exact same procedure can also be used with the critical hydraulic gradient as the limit state with different input parameters applicable to the underseepage analysis.

Once the standard deviation and expected value for the factor of safety are known, the coefficient of variation (COV) for the factor of safety may be calculated and then used in Equation A-4.12 to compute the reliability index. Given the reliability index, β, the probability of failure is calculated using the built-in function NORMSDIST in Microsoft Excel. This function uses the reliability index as the argument allowing for the probability of failure to be computed as:

$$P_f = 1 - \text{NORMSDIST}\ (\beta) \qquad \text{Equation A-4.21}$$

A-4.4.3 Uncertainty Analyses

A-4.4.3.1 General

Risk-based analyses for the CID-KS Levee Unit were performed for the existing conditions. In these reliability analyses, geotechnical uncertainties were assessed by determining probability distributions for the blanket thickness and soil material properties for typical levee sections representative of the CID-KS Levee Unit.

Two types of geotechnical failures were analyzed:

1. A slope failure, defined as failure of the landside embankment slope resulting in water from the river flowing to the landside areas of the levee resulting in economic damages to the interior.
2. An underseepage failure, defined by excessive seepage initiating a levee failure and resulting in economic damages to the interior. Geotechnical failures may occur when river stages reach elevations at or below the top of levee. The geotechnical uncertainty analysis was performed only for the economic model and does not assess whether the levee unit meets or exceeds past or present design criteria.

The probability of failure of the levee is also conditional on the uncertainties associated with the hydrologic and hydraulic aspects of determining the water surface profile during a flood. These uncertainties can be combined with the geotechnical uncertainties and used in the HEC-FDA program. This is performed for economic purposes through the development of a relationship between the probability of failure of the levee and the height of water on the levees.

A-4.4.3.2 Probabilistic Underseepage Analysis

The actual conditions indicative of an underseepage failure are highly speculative. The underseepage analysis included in ETL 1110-2-556 - Appendix B uses a threshold value of gradient factor of safety of 1.0 to define failure. A gradient factor of safety of 1.0 reflects a condition where floatation of particles theoretically begins and seepage and boils can first physically occur, however it is not necessarily a condition indicative of having certain levee failure. Observations during the Flood of 1952 on the Missouri River are shown in Table A-4.2. The table shows the relation between observed field performance and calculated factors of safety. From the observations it can be seen that somewhere between a factor of safety of 0.55 and 0.80, undesirable seepage reaches a point where a failure could occur without outside intervention in the form of flood fighting. In an effort to define a condition more representative of actual levee failure due to underseepage for this study, a gradient safety factor of 0.70 was utilized as a threshold value for when certain levee failure is likely to occur. The chosen threshold value of gradient factor of safety of 0.70 falls within the "transition" zone in Table A-4.2 between tolerable seepage and objectionable seepage. In the probabilistic underseepage analyses, a failure gradient (i_f) was calculated as:

$$i_f = \frac{i_c}{FS} = \frac{0.84}{0.70} = 1.23 \qquad \text{Equation A-4.22}$$

where i_c is the critical gradient and FS is the gradient safety factor. The factor of safety that defines failure was used to define the failure gradient in Equation A-4.22 and the limit state in Equation A-4.13.

TABLE A-4.2
Observations of Seepage Conditions During 1952 Flooding on the Missouri River at the Kansas Citys Flood Control Project

Computed Safety Factor at Flood Crest	Seepage conditions during flood Crest
Less than 0.55	Objectionable seepage: major flood fight; boils requiring sandbagging
0.55 to 0.80	Transition zone
Greater than 0.80	Tolerable seepage: distributed seepage, pin boils

The Kansas City District method of estimating the hydraulic gradients due to underseepage is slightly different than the method described in the EM 1110-2-1913. It is based on the findings made at the Missouri River Division Conference held by the Corps of Engineers in 1962 in Omaha. The underseepage analysis was based on experience during the flood event in 1952 along the Missouri River. The main differences in the Kansas City District method are:

1. The Kansas City District Method uses permeability ratios (See Table A-4.3.) related to differing material types of the blanket material instead of using actual horizontal and vertical permeabilities.

2. The Kansas City District Method assumes an infinite landside blanket in the analysis.

3. The Kansas City District Method does not use a transformed thickness for the soil stratum considered as EM 1110-2-1913 allows, instead, a representative permeability ratio is applied to the overall blanket thickness.

TABLE A-4.3
Permeability Ratios for Blanket Material Based on Material Type

Blanket Material	Assumed Permeability Ratio
SM	100
ML	200-400
ML - CL	400
CL	400-600
CH	800-1000

Additional information concerning the underseepage analysis for the Kansas City procedure can be found on the District's website at: http://www.nwk.usace.army.mil/Portals/29/docs/construction/underseepage1.pdf.

The critical section for an underseepage failure along the CID-KS Levee Unit was chosen by calculating the expected value of the factor of safety with respect to hydraulic gradient at the toe of the levee for the entire unit. The reach with the lowest expected factor of safety was chosen for a risk analysis.

In the probabilistic analyses of underseepage using the Kansas City District method, three random variables were considered: blanket thickness, the permeability ratio and thickness of the aquifer.

Using existing subsurface information, it was assumed that the COV of the blanket thickness and thickness of the aquifer was 25 percent and 15 percent, respectively. These values for COV are deemed appropriate for the level of information available.

Using the published value given in ETL 1110-2-556, it was assumed that the COV of the permeability ratio was 40 percent. The permeability ratios used in the analyses followed the Kansas City District Guidance based on the type of material making up the blanket layer. In the existing conditions phase of the study the permeability ratios used in the underseepage analyses were based on material descriptions obtained from historical borings information from the CID-KS Unit. Table A-4.3 lists the permeability ratios.

The underseepage analyses are then performed using the expected values of the random variables and plus and minus one standard deviations at different river levels. Using the log normal distributions and the limit state function for underseepage, a probability of failure can be developed for each river level at the critical locations.

A-4.4.3.3 Probabilistic Slope Stability Analysis

The conditions leading to a stability failure are less uncertain than those of an underseepage failure. A threshold value of stability factor of safety of 1.0 to define a slope failure is nearly universally accepted. The assumptions made for the slope stability component of the risk-based analysis allowed the evaluation to be more specific as to the magnitude of the failure and the actual consequences associated with that type of failure. The slope stability analyses assumed that the failure surface should be of significant magnitude to remove a major portion of the levee allowing the interior of the levee unit to flood.

The critical section for the stability analysis was chosen based on levee height and side slope steepness. The section with the tallest levee height and steepest side slopes was chosen for the probabilistic analysis.

Each zone of material making up the critical cross section of the levee was considered homogenous. The zones were comprised of three areas: the foundation sands, the blanket materials, and the embankment material. The foundation sand strengths were considered constant in the analysis. The piezometric surface through the levee cross section was simplified and considered to be in a steady state condition. The model that was used assumed that the water surface entered the slope at the point on the riverside where the river intersected the upstream slope face. The piezometric surface then continued in a linear path to the landside levee toe.

The soil strength parameters considered in the existing conditions analysis were modeled with drained strengths because steady seepages conditions were considered. The mean values and coefficients of variations were computed from raw data located in Design Memorandum No 4. The raw data used in this study was taken from consolidated drained direct shear tests performed for the 1962 Modification of the CID-KS Unit. The effective stress failure envelopes for normal effective stresses less than 2000 psf were used to characterize the strengths of the soils. This was done because the "working load" effective stresses in the embankment and foundation materials are generally near, or less than, this value during flood conditions.

The materials evaluated were designated as either foundation blanket material or embankment fill material. Based upon available laboratory test data, along with the results shown in Table A-4.4, it was determined that the blanket had an expected value ($E[\bar{\phi}]$) of 26° with a coefficient of variation ($COV_{\bar{\phi}}$) of 12 percent, and the embankment had an expected value ($E[\bar{\phi}]$) of 32° with a coefficient of variation ($COV_{\bar{\phi}}$) of 11 percent. Cohesion (c) was assumed to be zero with no variation for both materials.

The pore pressures developed in the blanket material were determined from the hydraulic gradient calculated at the base of the blanket material due to underseepage. The hydraulic gradient line was based on the output from the underseepage analysis using the Kansas City District Method. Assuming that the elevation head datum is at the same elevation as the base of the blanket material, the pore pressure (u) at a point along the base of the blanket material would be equal to the distance from the hydraulic gradient line (h_p) to the base of the blanket multiplied by the unit weight of water (γ_w). The mathematical relation can be stated as follows:

$$u = h_p * \gamma_w \qquad \text{Equation A-4.23}$$

For points within the slope, the pore pressure at the top of the blanket are calculated as the distance from the phreatic surface to the top of the blanket (h_p) multiplied by the unit weight of water (γ_w) (as in Equation A-4.23). The pore pressure at the base of the blanket was calculated using the distance from the hydraulic gradient line as the pressure head (h_p) in Equation A-4.23. A linear interpolation between these two pore pressures would give the pressure distribution through the blanket material used in the slope stability analysis.

The embankment was assumed to be homogenous and impervious, even though it is comprised of impervious and random zones. This was done to simplify the analysis and due to the fact the random material is mostly comprised of impervious material.

TABLE A-4.4
Effective Stress Strength Data Used for Risk and Reliability Analyses
Embankment and Foundation Materials

Material	Boring	Sample	Soil	τ (tsf)	σ (tsf)	ϕ (degrees)
Blanket	U-408	WAX-3T	ML	0.48	1.0	25.6
Blanket	U-408	WAX-3C	CH	0.49	1.0	26.1
Blanket	U-409	WAX-2	CH	0.63	1.0	32.2
Blanket	U-409	WAX-5	CH	0.55	1.0	28.8
Blanket	U-409	WAX-7	CL	0.56	1.0	29.2
Blanket	U-410	WAX-3	CH	0.46	1.0	24.7
Blanket	U-410	WAX-6	CH	0.49	1.0	26.1
Blanket	U-410	WAX-9	CH	0.47	1.0	25.2
Blanket	U-411	WAX-1	ML	0.50	1.0	26.6
Blanket	U-411	WAX-3	CH	0.48	1.0	25.6
Blanket	U-411	WAX-6	ML	0.34	1.0	18.8
Blanket	U-411	WAX-9	CH	0.56	1.0	29.2
Blanket	U-412	WAX-5	CH	0.46	1.0	24.7
Embankment	A-413	SK-1	CL	0.67	1.0	33.8
Embankment	A-413	SK-2	CL	0.61	1.0	31.4
Embankment	A-414	SK-1+2	CL-ML	0.65	1.0	33.0
Embankment	A-415	SK-1	CH	0.5	1.0	26.6
Embankment	A-415	SK-2	CL	0.7	1.0	35.0
Embankment	A-417	SK-1+2	CL	0.53	1.0	27.9
Embankment	A-420	SK-1+2	CL	0.72	1.0	35.8

The slope stability analyses are carried out in the same manner prescribed in ETL 1110-2-556. Utilizing the slope stability program UTEXAS 4 (using Spencer's Method), an initial circular search is performed using the expected values (means) for the random variables considered in the analysis. In order to determine a surface that would mobilize a large portion of the embankment that would lead to a catastrophic failure, a series of single surface searches are performed to locate the critical surface. The failure surface is forced to include a major portion of the embankment to model a catastrophic failure that would cause interior flooding. Using this boundary condition, the failure would be of significant magnitude to inundate the levee interior instead of assuming a progressive slope failure from the landward levee toe.

An initial run in the UTEXAS 4 program is made using the expected values E[$\bar{\phi}$] for each of the different material types. The factor of safety (FS) obtained from this analysis gave the expected value for the factor of safety E [FS]. The failure surface obtained from this initial run was then considered the critical surface. The remaining series of runs were made at plus and minus one standard deviation of the expected values for strength along the critical surface defined in the initial run. As each material property was changed, a resulting factor of safety was computed. The variation resulting in each change for that particular material type can then be used in the Taylor Series Approximation. Using the probabilistic methods described previously, a probability of failure could be determined for a specific river elevation. The procedure was then repeated for various river levels and a probability curve was computed based on slope stability relationships with river levels.

A-4.4.4 Results for the Reliability-Based Analyses of the Kansas Citys – Missouri and Kansas Flood Risk Management Project

A-4.4.4.1 Underseepage Results

The critical section for the CID-KS Levee Unit with respect to an underseepage failure was computed to be the old "stockyard" area at approximately Station 85+00 landward of the large retaining wall in a reach containing 10 relief wells. This section was chosen as the critical section because it had the lowest expected value for factor of safety (1.05) for the entire unit. The "stockyard" area is comprised of a small levee section on top of a large fill supported by a landward retaining wall. The retaining wall is up to 20 ft tall. A series of actively pumped fully penetrating relief wells exists near the landward toe of the retaining wall. The well flow is carried to the Stockyard No 3 Pump Station by a header system. For the purposes of the existing conditions analysis the wells were assumed to have lost 20% of their efficiency.

The typical section used in the analysis consisted of 22.5 ft of driving head, and an expected value for blanket thickness in the slot of 20-ft. The expected value for permeability ratio and foundation sand depth is 400 and 50-ft, respectively.

The calculation necessary to determine the probability of failure for the slot area that includes the uncertainty in the well flows would be computationally intense. So the probability of a levee failure due to piping in the "stockyard" area was calculated with some deviations from the method described in the probabilistic underseepage analysis discussion due to the relief wells in the "stockyard" area. First, the expected value of the factor of safety was calculated, including the effects of the relief wells and interior ponding, for varying river stages for the slot area. The expected value of the factor of safety is provided in Table A-4.5. To approximate the probability of failure from the calculated expected value of the factor of safety considering the relief wells, a relation between expected value of the factor of safety and probability of failure was developed using the methods described in the probabilistic underseepage analysis discussion that does not consider the relief wells. The statistical parameters described above for the coefficient of variances and threshold values were used in the determination of the relation. The relation, shown in Exhibit A-4.8 at the end of this chapter, was then used to determine the probability of failure using the calculated expected value of the factor of safety.

The probability of a levee failure due to piping in the "stockyard" area for the existing condition is shown in Exhibit A-4.9 at the end of this chapter. At the maximum river level, during steady state seepage conditions, the probability of failure is 4.5%. It should be noted that the probability of failure due to a piping failure in the "stockyard" area increases slightly if the relief wells are not actively pumped and the wells discharge at the ground surface. It is only a small increase in the probability of failure if the wells are not actively pumped because the discharge elevations are generally within a couple feet of the ground surface.

TABLE A-4.5
Expected Value of Factor of Safety for Piping – Station 85+00

River Elevation (ft)	Total System Relief Well Flow (cfs)	$FS_{iexpected}$
753	4.0	1.69
755	4.9	1.5
757	6.1	1.35
759	7.2	1.22
761	8.3	1.12
762.5	9.1	1.05

A-4.4.4.2 Stability Results

The critical section for the CID-KS Levee Unit with respect to slope stability was located at approximately Station 68+00. This section was chosen as the critical section due to the levee height and levee side slopes. There is a short reach from Station 74+00 to 74+50 with a localized taller section, but three dimensional effects likely keep it from being the critical section and the Station 68+00 section was used as the critical section.

The levee at Station 68+00 has a typical cross section of a 15-ft high levee with a side slope of 4:1 (horizontal to vertical) on the riverside, a crest width of 10-ft, and a net side slope of 3.5:1 (horizontal to vertical) on the landward side. Based on stability analysis performed for the Armourdale feasibility N500+3 raise design, the section will meet existing design criteria and have negligible risk from stability failure. Therefore a risk analysis was not performed. The analyzed Armourdale section was a 16.5-ft high levee with a 4:1 landward side slope on similar foundation conditions as found in the CID-KS reach. The foundation strength for Armourdale design is similar to the mean strength for the CID-KS levee that would be used in a risk analysis.

A-4.4.5 Summary

The geotechnical existing conditions analysis was performed to identify the critical sections from a geotechnical perspective and determine their probability of failure. The probabilistic analyses performed for this study were modeled with guidance given in ETL 1110-2-556 "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies" (28 May 1999).

Two modes of unsatisfactory performance were considered at various river stages-underseepage and landside slope stability under a steady state seepage condition. Where enough information was present, the probabilistic parameters needed for each of the variables as calculated. If little or no raw data was available, assumptions were made based on work done by others in the field of geotechnical risk-based analysis.

A-4.5 DETERMINISTIC ANALYSIS METHODS FOR DESIGN OF NEW FEATURES

A-4.5.1 Slope Stability Criteria

For the CID-KS Unit of the Kansas City Levees Feasibility Study Phase 2, no stability analyses were performed on specific CID-KS levee sections. Since the foundation conditions and the river loading conditions were very similar to the Armourdale Unit which exists along the left bank of the same stretch of the Kansas River, the results of the analyses from Armourdale were applied to the CID-KS levee sections. Comparison criteria included height of levee, thickness of the natural blanket, river loading and hydraulic gradient. The criteria used in the Armourdale analyses (same as would have been used for the CID-KS Unit) is discussed below. It is recommended that during design the stability of the levees in CID-KS Unit be fully investigated.

The criteria used for the slope stability analysis was from Engineering Manual 1110-2-1913, Design and Construction of Levees, dated April 2000. The engineering manual lists the following minimum requirements in Table A-4.6 with respect to a deterministic slope stability analysis:

TABLE A-4.6
Minimum Factors of Safety

Loading Condition	Minimum Factor of Safety
End of Construction	1.3
Steady Seepage	1.4
Rapid Drawdown	1.0 to1.2*

*Lower factors of safety may be appropriate when the consequences of failure, environmental damage, and/or economic losses are small.

The rapid drawdown stability analysis was not performed due to the lack of required shear strength parameters for the two stage analysis. Additional drilling and testing will be required as part of PED to determine the shear strength parameters for this analysis.

For levees in an urban area, rapid drawdown failure could be significant in terms of economics, not only for the temporary loss of protection but also for repairs to the levee. It is recommended that a factor of safety of 1.2 be used for this failure condition. The engineering manual does not specify the water levels for the different loading conditions, so the following, in Table A-4.7, was assumed:

TABLE A-4.7
Water Loading Conditions

Loading Condition	Water Level for Stability Analysis
End of Construction	Water at Top of Natural Blanket
Steady Seepage	Water at Top of Protection, Riverside
Rapid Drawdown	Water at Top of Protection, Stage 1 Water at Riverside Toe of Levee, Stage 2*

*Or landside ground elevation, whichever is lower

Guidance was published by the HQUSACE in April 2007 with respect to Hurricane Protection System slope stability design criteria guidance. The document was published in the form of a Memorandum to the Commander, Mississippi Valley Division, and intended for use during levee rehabilitation in Southeast Louisiana. The revised design criteria is based on criteria presented in EM 1110-2-1902 Slope Stability, dated Oct 2003, for new embankment dams. The original criteria are consistent with that presented in Table A-4.6. The new guidance suggests a factor of safety of 1.5 (if the site conditions are "well defined") for what is called the "extreme hurricane" condition, when steady state conditions are expected to develop with water at the top of protection. This is an increase in factor of safety from what was used for this feasibility study. Though the published criteria are currently related to hurricane loadings, it could easily be transferred to all levees in the future. It is suggested that slope stability criteria be reviewed and revised as necessary during PED. If increased factors of safety are required for the CID-KS Unit, the implications would be additional cost and required real estate for expansion of stability berms.

A-4.5.2 Underseepage Criteria and Analysis

The current Corps of Engineers guidance on underseepage is contained in Engineering Technical Letter (ETL) 1110-2-569. The ETL recommends using all definitions, design equations, and procedures in Engineering Manual (EM) 1110-2-1913 except as noted within. The greatest deviation from the EM is the requirement for a maximum hydraulic gradient through the landside blanket at all points landward of the levee of 0.5, which provides for a factor of safety with respect to hydraulic gradient (FS_i) of approximately 1.6. For the design of future conditions alternatives of the CID-KS Unit, the criteria shown below was used to determine whether underseepage control measures are necessary.

With water at the top of line of protection:

- FS_i equal to, or greater than, 1.6 – No underseepage control measures are necessary.
- FS_i less than 1.6 – Design underseepage control measures to achieve a FS_i = 1.6.

The general procedure outlined in EM 1110-2-1913 Design and Construction of Levees was used to calculate the factor of safety with respect to hydraulic gradient for the natural blanket, and to calculate the excess head at the landside toe (assumed to be acting at the bottom of the blanket) of the line of protection. The variations from EM 1110-2-1913 used in the analysis is as follows and discussed previously in this chapter:

1. The use of permeability ratios relating to different material types for the natural blanket, as opposed to actual horizontal and vertical permeabilities.

2. The assumption of an infinite landside blanket.

3. No blanket thickness transformation is performed.

The general procedure outlined in EM 1110-2-1914 Design, Construction, and Maintenance of Relief Wells, Figure 5-3 was used to analyze and design all relief well systems. The variations from EM 1110-2-1914 used in the analysis and design are:

1. The excess head computed at the landside toe was used as the net head on the system of wells instead of full driving head. This was done because the procedure outlined in Figure 5-3 assumes an impervious blanket. However, a semi-pervious blanket was assumed for the underseepage calculations.

2. An efficiency reduction factor of 0.8 was applied to the expected well flows. This was done to account for the reduction in efficiency with time of the relief wells. An efficiency factor of 0.8 was chosen as EM 1110-2-1914 requires remedial action once a loss of 20% in specific capacity of a well is observed from pumping test.

A-4.6 N500+3 STABILITY ANALYSES

A-4.6.1 Sections Analyzed

For the CID Kansas Unit, the design team utilized the analyses performed for the Armourdale Unit. The foundation conditions on the two units are similar and this approach was deemed adequate for a feasibility level of analysis. Several protection raise configurations to raise the level of protection to an N500+3 event on the Kansas River were considered. The method for computing the river stage for this event is discussed in the Hydrology and Hydraulics chapter in the Phase 1 Feasibility Report. The top of the proposed raise was set approximately equal to the river stage for the N500+3 event.

The raise configurations that were considered are as follows:

1. Landside earth fill raise
2. T-Wall on levee
3. Floodwall

All levee sections will maintain a 10-ft crest width to maintain the current level of vehicle access. Floodwall stability from a geotechnical standpoint is addressed at the end of this section.

To begin the evaluation process, a site visit was made to CID Kansas to make an initial attempt to identify the most appropriate raise configuration along the entire unit on a reach by reach basis. The initial appraisal was that levee sections remain levee sections and floodwalls remain floodwalls. Next the levee sections were evaluated by considering the existing levee height, height of proposed raise, thickness of the natural blanket, and the results of the underseepage analyses. These details were required to compare to the Armourdale analyses.

The computer program UTEXAS4, developed by Stephen G. Wright of the University of Texas at Austin, was used to perform the Armourdale stability analyses. The selected analysis method was Spencer's method, which is a limit-equilibrium approach that satisfies both force and moment equilibrium. The program has the ability to "search" for the critical failure surface with the lowest factor of safety for the given input parameters. As stated previously, only the end of construction and steady seepage loading conditions were analyzed. Steady seepage conditions controlled the final section dimensions for all sections analyzed. Potential rapid drawdown failure of the riverside slope will also have to be evaluated after additional geotechnical laboratory testing can be performed to determine the necessary strength parameters required for this analysis. It is recommended that stability analyses be performed for the CID Kansas levee sections during PED.

A-4.6.2 Landside Earth Fill Raise

A landside earth fill raise is the preferred raise configuration due to the low cost and ease of construction, and was proposed at all locations where an existing levee existed. Two sections from the Armourdale analyses were utilized for application to CID Kansas, Section 1 and Section 2 as shown in Exhibit A-4.10 in the supplemental exhibit section. The levee sections will be raised by maintaining the riverside slope and shifting the levee centerline landward. The analyzed Armourdale sections are described below.

Section 1 is the shorter section of the two. However, it had the highest piezometric levels in the foundation of the reaches where the landside fill was proposed at Armourdale. The proposed height of the levee for the N500+3 raise was 16.5-ft. The stability analysis indicated an acceptable cross section requires a 1V on 4H landside slope. A drainage layer was added under the new landside fill to improve the internal seepage conditions in the embankment.

Section 2 was the tallest proposed section for the landside raise configuration at Armourdale. The proposed height of the levee is 20-ft. The general approach for analyzing this section was the same as for Section 1. For this section to meet the minimum factor of safety two stability berms were required to be added to the basic levee section developed for Section 1; a 20-ft wide, 8-ft tall berm with a 1V on 4H slope and a shorter 10-ft wide, 4-ft tall berm with a 1V on 3.5H slope. Again, a drainage layer was added under the new landside fill to control the internal seepage conditions in the embankment.

Along most of the CID Kansas Unit the proposed levee sections fall at or below the height of Section 1. The exception is the levee section in the Kemper Arena area, Station 77+25 to approximately Station 100+00, where Section 2 was applied. This was due to removal of the existing retaining wall along the landside of the section. A summary of existing levee stationing, existing heights, proposed raises, and recommended section is provided in Exhibit A-4.11 at the end of this chapter.

There is one reach of levee section that is adjacent to an existing HTRW site. This reach is between approximately Stations 40+00 to 51+00, and is currently occupied by Advantage Metals Recycling. The proposed raise in this section will have to be modified so no additional fill will be placed landside of the current levee toe. See Section 3 in Exhibit A-3.13 in the previous chapter for reference.

A-4.6.3 T-Wall on Levee

Because of the high cost of modifying or replacing floodwalls for the proposed protection raises, the unit was evaluated for the possibility of replacement of the existing floodwall reaches with a T-wall on levee section that was utilized on the Armourdale Unit. The evaluation identified only one reach where this section could potentially be applied: between approximate Stations 26+70 and 40+30. This proposed section is provided in Exhibit A-4.12 in the supplemental exhibits section. The final selection of the proposed raise alternative in this reach will be based upon the cost of the adjacent real estate relative to the cost of a floodwall replacement. For the sake of the feasibility study cost estimate, floodwall modifications were assumed.

A-4.6.4 Floodwall

In addition to the T-wall on levee section evaluated in the above paragraph, a second floodwall section was investigated for replacement with a levee section. This reach was from approximately Stations 102+70 to 115+00. An adjacent roadway required a retaining wall along the downstream toe which affected the stability of the section. Because of this, the option was not pursued further and this reach will remain a floodwall section.

A-4.6.5 Geotechnical Floodwall Evaluations

CID Kansas has several sections of existing floodwalls that are discussed in more detail in the Structural Engineering Appendix. This section documents the geotechnical work related to the analysis of these walls. The floodwalls were originally constructed in the 1930's, and then raised as part of the 1962 Modification. The N500+3 proposed modifications include mostly raising and modifying existing floodwalls by adding new landward piles, base extension, and structural reinforcement. This is possible because the existing timber piles supporting the floodwall were investigated and found to be in good condition with no signs of decay or deterioration. Again, because of the similarity of the foundation conditions with the Armourdale Unit, pile designs from Armourdale were typically applied to the CID Kansas floodwalls. Refer to Chapter 3 of this appendix detailed discussion of the analyses for the Armourdale Unit. For wall foundation modifications see the structural chapter of this Engineering Appendix.

A-4.7 N500+3 UNDERSEEPAGE ANALYSES AND CONTROL FEATURES

A-4.7.1 Subsurface Information

The natural blanket was characterized using subsurface information obtained from Design Memorandum No 4. The memorandum contains the results of an extensive subsurface investigation that was performed for the design of the 1962 Modification of the CID-KS Unit. The subsurface investigation was used to identify the soils present in the foundation of the line of protection, and to establish their geotechnical parameters. This information was used to determine blanket thickness and composition which was used in the underseepage analysis. The CID-KS Unit has a foundation blanket varying between 10-ft and 40-ft thick, consisting of silts, clays, fill (less than 30% fines) and discontinuous sand lenses. Underlying the foundation blanket is between 50-ft and 60-ft of sand before bedrock is encountered. Overlying the foundation blanket is between 0 and 15-ft of fill which was considered pervious in nature if it contained less than 30% fines and subsequently not included in the blanket thickness in the underseepage calculations. Pervious fill overlying the foundation blanket was considered to be saturated, and the landside ground surface was used as the tailwater elevation. This is a generally conservative assumption and is valid if the fill does not meet filter criteria with the adjacent blanket condition. If it is found later that the fill does meet filter criteria with the adjacent blanket material there could be significant cost savings by reducing the number of new relief wells required.

A-4.7.2 N500+3 Underseepage Analysis

Raising the CID-KS Levee Unit to a N500+3 level of protection increases the water pressures in the foundation sands, which in turn increases the hydraulic gradient through the natural blanket material. For the underseepage analysis, the entire CID-KS Levee Unit was divided into reaches of similar proposed protection height, blanket thickness, blanket composition, aquifer thickness, overlying fill thickness, and seepage entrance conditions. The factor of safety with respect to hydraulic gradient through the blanket was calculated for each of these reaches at the toe of the line of protection, and other known critical areas such as building foundations and low areas as necessary. If the calculated factor of safety with respect to hydraulic gradient was calculated to be greater than 1.6 at all locations landward of the line of protection, no remedial measures were proposed. If the calculated factor of safety with respect to hydraulic gradient was calculated to be less than 1.6, remedial measures were proposed that would achieve a factor of safety with respect to hydraulic gradient of 1.6 at all locations landward of the line of protection. The design condition is to have a factor of safety with respect to hydraulic gradient equal to, or greater than, 1.6 at all locations landward of the line of protection toe. Exhibit A-4.13, at the end of this chapter, shows the calculated factor of safety with respect to hydraulic gradient for the entire CID-KS Levee Unit, as well as the parameters used to calculate the factor of safety. If a remedial measure is required to bring the reach into criteria, it is noted on the exhibit.

A-4.7.3 Underseepage Control Requirements

The reaches outlined in detail in this section are either at the minimally acceptable factor of safety with respect to hydraulic gradient without remedial measures, or require remedial measures to increase the factor of safety to the minimally acceptable factor of safety with respect to hydraulic gradient. Remedial measures considered were relief wells and area fills. If cut off walls become more economical to construct, they may be considered in lieu of relief wells during PED. All existing underseepage control features on the CID-KS Unit outlined previously are assumed to remain in place as functional features, except as noted in the following discussion. All relief wells are considered to be fully penetrating.

Station 32+00 to 38+00

The flood protection in this reach consists of a floodwall that is approximately 7-ft in height that was constructed on an earthen fill section. The landside ground surface beyond the earthen fill section decreases in elevation in an up-station direction, and there is a retaining wall at the toe of the fill section between Stations 35+00 and 38+00. The total driving head ranges from 12 to 18-ft from the top of the floodwall to the low area landward of the landside retaining wall. The calculated factor of safety with respect to hydraulic gradient in this reach ranges from 1.15 to 1.55 with water at the N500+3 elevation. Both a relief well solution and an area fill solution were analyzed to alleviate the underseepage concerns in this area caused by the proposed raise.

Area Fill

The low area landward of the earthen fill section and retaining wall must be raised to a minimum elevation of 748 to meet underseepage criteria with an impervious fill to meet underseepage criteria. This is an increase in elevation between 0 and 5-ft over an area approximately 550-ft long and extending approximately 200-ft landward of the earthen fill section toe. The fill is needed between approximate Stations 32+50 and 38+00. The area is currently paved, with a drop inlet for surface drainage. The interior drainage will have to be redesigned so the area is not adversely affected by the fill. The existing pavement will be removed prior to fill placement and the area paved once the fill and final grading are complete. There is one small building located near the limits of the proposed fill. Once an accurate survey has been completed, the fill details around the building can be finalized. If a pervious fill is used for the area fill, the minimum fill elevation is 749. Exhibit A-4.14 show the area fill calculations at the end of this chapter.

Relief Wells

The installation of 12 surface discharge relief wells must be used to meet underseepage criteria. The wells will need to be spaced between 12 and 50-ft apart and would be placed at the landside toe of the earthen fill section (landward of the retaining wall). The

12 wells will discharge approximately 13 cfs of underseepage to the ground surface. The relief well flow will drain to an existing surface drainage inlet landward of the retaining wall and will be directed toward the Mistletoe pump station. The Mistletoe pump station will have to be modified to handle the flow or landward ponding will occur. Exhibit A-4.15 show the relief well calculations at the end of this chapter.

The area fill alternative is the proposed alternative to bring this reach into compliance with the underseepage criteria. It is a highly reliable solution with a relatively low construction cost. Additionally, there are no future operation and maintenance requirements or costs associated with an area fill.

Station 51+00 to 97+00

This reach consists of an earthen levee section, with the exception of the gap structure on top of railroad fill near Station 75+00. Downstream of the gap structure the levee section will be between 13 and 20-ft tall after the proposed raise, generally increasing in an up-station direction. Downstream of the gap structure there is a low area adjacent to the levee and beyond the low area the landside ground surface is several feet higher. Upstream of the gap structure the levee section will generally be less than 10-ft tall, however there is a large retaining wall landward of the levee which is up to 20-ft tall creating a large head differential across the levee section. Landward of the landside retaining wall is the old stockyards area which is a mix of open ground and paved parking areas. There is an existing series of 10 relief wells upstream of the gap between Stations 79+00 and 97+00. The relief well flow is diverted to the Stockyard No 3 pump station downstream of the gap near Station 74+00. The calculated factor of safety with respect to hydraulic gradient in this reach ranges from 0.8 and 1.78 with water at the N500+3 elevation with the existing relief well system only. Both a relief well solution and an area fill solution were analyzed to alleviate the underseepage concerns in this area. If the area fill solution is implemented, the relief well system and the Stockyard No.3 Pump Station should be abandoned and/or removed.

Area Fill

The low area landward of the levee downstream of the gap will need to be raised to elevation 751 at the toe of the levee between Stations 63+00 to 74+75 to meet underseepage criteria. The area fill should slope landward at 1V on 100H to provide for surface drainage. This is an increase in elevation between 3 and 5-ft at the toe of the levee and the fill will extend 200 ft landward of the levee toe where it will intersect the existing ground surface at approximate elevation 749. To accommodate the fill the Stockyard Pump Plant near Station 74+00 will have to be abandoned. The Stockyard Pump Plant has many pipes which discharge into the wet well. These pipes should be fully investigated prior to the Stockyard Pump Plant being taken offline to ensure no existing drainage condition is worsened. If a pervious fill is used for the area fill, the minimum elevation for the fill is 753.

The area landward of the retaining wall upstream of the gap will need to be raised to elevation 749 at the landward retaining wall between Stations 77+00 and 94+50 to meet underseepage criteria. The area fill should slope landward at 1V on 100H to provide for surface drainage. This is an increase in elevation between 1 and 9-ft at the retaining wall and the fill will extend 300 ft landward of the retaining wall where it will intersect the existing ground surface at elevation 746. The fill area is mostly open ground except for a parking lot for the Sprint complex. The final area fill design and layout will have to accommodate the Sprint complex's parking needs. The existing relief wells in this portion of the reach should be properly abandoned. If a pervious fill is used for the area fill, the minimum elevation for the fill is 753.

The area fill was also analyzed as an underseepage berm (where the berm width is included in the levee width) to ensure a factor of safety of at least 1.6 with respect to hydraulic gradient will exist at the toe of the "berm". Exhibit A-4.16 show the area fill calculations at the end of this chapter.

Relief Wells
For the design of a relief well alternative, the existing well system was assumed to remain operational and intact and proposed well locations were designed around the existing system. The existing 10 relief wells will have flows under the N500+3 condition similar to the current expected flows (14 cfs). The installation of 28 surface discharge relief wells (32 cfs) downstream of the gap and 27 surface discharge relief wells (28 cfs) upstream of the gap will be required in addition to the existing relief well system. The wells will need to be spaced between 25 and 200-ft apart, and placed at the toe of the levee downstream of the gap and the toe of the retaining wall upstream of the gap. The 55 wells will discharge approximately 62 cfs of underseepage flow onto the ground surface (32 cfs downstream of the gap and 30 upstream of the gap). The relief well discharge will collect in low lying areas landward of the levee, and eventually flow into existing drainage features. If landward ponding cannot be allowed, some provision for collecting the seepage and transporting it to the riverside of the levee will have to be implemented. Exhibit A-4.17 show the relief well calculations at the end of this chapter.

The area fill alternative is the proposed alternative to bring this reach into compliance with the underseepage criteria. It is a highly reliable system with a relatively low construction cost. Additionally, there is no additional future operation and maintenance requirements or costs associated with an area fill and existing operation and maintenance costs and existing requirements can be removed with the abandonment of an existing relief well system and pump station.

Station 107+00 to 116+00
This reach consists of a floodwall section. The floodwall will be approximately 15-ft tall after the proposed raise. The foundation blanket is overlain by between 5 and 13-ft of fill which was considered to be pervious in nature. The pervious fill was treated as saturated sand in the underseepage analysis and was not included in the blanket thickness. The calculated factor of safety with respect to hydraulic gradient in this reach ranges from 1.2 to 1.3 with water at the N500+3 elevation. Relief wells, an area fill, and removal of the pervious fill were originally considered for this reach. An area fill would be difficult due to the existing topography, existing surface drainage, lack of easily obtainable real estate, and the pervious fill overlying the natural embankment. It is likely that the pervious fill

continues for a significant distance landward, and removing and replacing it would create real estate and cost concerns. Relief wells can be installed with little impact landward of the line of protection, and were the only fully analyzed option for this reach to bring the area into accordance with criteria. The installation of 27 surface discharge relief wells near the landward toe of the floodwall will be necessary to achieve the required factor of safety. The wells will need to be spaced between 20 and 45-ft apart. The 27 wells will discharge approximately 18 cfs of underseepage flow onto the ground surface. The relief well discharge will flow into the low lying areas landward of the floodwall, and flow into existing drainage features towards the Kemper Arena Pump Station. If landward ponding cannot be allowed, some provision for collecting the seepage and transporting it to the riverside of the levee will have to be implemented. Exhibit A-4.18 show the relief well calculations at the end of this chapter. The recommended wells would not be needed if the overlying pervious fill meets filter criteria with the underlying natural blanket materials.

Station 127+00 to 168+00

This reach consists of a floodwall section. The floodwall will be between 14 and 19-ft tall in this reach. The foundation soils in this reach consist of up to 8 ft of pervious fill overlying a natural blanket of an average thickness of 24-ft. The pervious fill was treated as saturated sand in the underseepage analysis and was not included in the blanket thickness. Additionally, the bluff blocks the seepage in the aquifer and does not allow excess head to dissipate landward as when the blanket and aquifer are assumed to be infinite. The calculated factor of safety with respect to hydraulic gradient in this reach ranges from 1.1 to 1.6 with water at the N500+3 elevation. Only relief wells were analyzed for this reach to bring the area into accordance with criteria due to landward railroad tracks prohibiting any area fill alternatives. The installation of 76 surface discharge relief wells near the landward toe of the floodwall and 7 surface discharge relief wells near the bluff line is required to achieve the required factor of safety at all points landward of the floodwall. The wells will need to be spaced between 25 and 150-ft apart. The 83 wells will discharge approximately 108 cfs of underseepage flow onto the ground surface. The wells along the floodwall between Stations 127+00 and 143+00 will discharge directly at the landside ground surface. The wells along the floodwall between Stations 143+00 and 168+00 will discharge at the elevation of the ground surface approximately 30-ft landward of the floodwall which is several feet below the immediate landside ground surface. A discharge detail will have to be designed to allow this to occur. The relief well discharge will flood the low lying area between the floodwall and bluff. Eventually the relief well flow will flow towards the Kemper Arena Pump Station; however significant interior flooding will occur prior to the flow reaching the pump station. If landward ponding cannot be allowed, some provision for collecting the seepage and transporting it to the riverside of the levee will have to be implemented. Exhibit A-4.19 shows the relief well calculations at the end of this chapter. There were ongoing discussions about terminating the CID-KS Unit before station 168+00. Relief well options were extrapolated for the early termination options based on the calculations for the non-early termination option. These options are shown in the Maps section of the Feasibility Report. The number of relief wells required would be significantly less if the overlying pervious fill meets filter criteria with the underlying natural blanket materials.

A-4.8 EXPECTED SETTLEMENT OF DESIGN FEATURES

No calculations were performed to determine the expected settlement of the proposed line of protection raise for the N500+3 condition. This is because no consolidation test data was found to determine the appropriate parameters required for settlement calculations. For feasibility level design the following estimations were made for the proposed raise configurations:

- Floodwall, no settlement
- Earth fill raise on existing levee, 3 inches maximum settlement.

It is recommended that during PED that additional soil sampling and testing be performed so the consolidation characteristics of the foundation materials can be quantitatively determined and settlement analysis performed to refine any required overbuild.

A-4.9 RECOMMENDATIONS FOR PED PHASE

1. Slope Stability

    a. Since the Armourdale stability analyses were used to select the proposed raise sections for the CID Kansas Unit, it is recommended that the CID-KS sections be evaluated and refined during the PED phase.

    b. It is recommended that the criteria used for slope stability be evaluated. The existing criteria was used for the Feasibility level design, however due to Hurricane Katrina there was a lot of discussion about increasing factors of safety during this time, and some interim guidance had been published. By PED the criteria for flood risk management projects may be revised. Any revisions to slope stability criteria should be incorporated into the final design.

    c. Resolve which raise solution will be utilized for the section of protection where the floodwall could be replaced with a T-wall on levee section, Station 26+70 to Station 40+30.

    d. Investigate the existence of an internal embankment drain between approximate Stations 41+00 to 74+35.94. This drain may be integral to the stability of the section if the section is not reconfigured.

    e. Investigate the global stability of the landward retaining wall in the old stockyards area.

2. Underseepage

    a. It is recommended that the levee unit be revisited for additional features, such as pits and low spots which need special attention with respect to the underseepage analysis that were not analyzed as a part of the feasibility study.

b. It is recommended that all existing relief wells to remain in use be pump tested and inspected to ensure required well flows can be achieved and the wells are in adequate condition. If the wells need to be replaced, the proposed relief well systems may be able to be refined and/or economized.

c. It is recommended that any changes in Corps of Engineers (or local district) guidance which governs underseepage analysis methods or criteria be captured during final design.

d. It is recommended that drilling and sampling be performed to determine if filter criteria is met between the overlying pervious fill and natural blanket materials. If filter criteria is met, significant cost savings could be realized in removal of planned relief wells.

3. It is recommended that a drilling and testing program be implemented to verify gaps in the existing data and to meet all criteria regarding subsurface investigation intensity. Those include (at a minimum):

   a. Borings and other subsurface investigations necessary to meet the requirements in ETL 1110-2-56 Design Guidance For Levee Underseepage

   b. Soil Strength Testing.

      i. Undrained strengths for the fill material and the blanket materials both under the existing levee sections and in the natural blanket outside the levee footprint.

      ii. R-bar triaxial testing on the fill section and the natural blanket materials to develop drained and undrained shear strength parameters needed for steady state and rapid drawdown analyses

      iii. Consolidation testing in reaches to receive fill for purposes of settlement estimation.

4. Recommend a full topographic survey in the critical zone of the line of protection, including all the way to the riverbank to feasibility study topographical assumptions can be verified.

5. Attempt to provide unrestricted vehicle access along the entire length of the line of protection for inspection and flood fighting purposes.

6. Recommend evaluating the impact of ground discharging relief wells on the interior drainage. The quantity of expected discharge from proposed wells for the N500+3 conditions would indicate that interior flooding could be a significant problem if the flow is not specifically handled.

7. A ground water study should take place in the area of any proposed cutoff wall to ensure local water interests will not be affected.

8. Cost estimates for relief well systems and cut off walls should be revisited during final design to ensure the more cost effective method is chosen.

A-4.10 REFERENCES

1. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial District - Kansas Unit, Volume I, Dated 1980.

2. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Centrail Industrial District Unit Kansas Section, Volume I, Appendix I, Dated 1950 - 1955.

3. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial District Unit Kansas Section, Volume II, Appendix I, Dated 1980.

4. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial Unit Missouri Section, Volume I, Dated 1981.

5. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial Unit Missouri Section, Appendix I, Dated 1948 - 1955.

6. Ang, A., and Tang, W., (1975), *Probability Concepts in Engineering Planning and Design* (Vol. I). New York: John Wiley & Sons, Inc.

7. Baecher, G. B., & J. J. Christian (2000), "Uncertainty, Probability, and Geotechnical Data," paper presented at Performance Confirmation of Constructed Geotechnical Facilities, ASCE, Amherst, MA; April 9-12.

8. Hunt, R. E., (1986), *Geotechnical Engineering Analysis and Evaluation,* New York: McGraw-Hill Book Company.

9. Reese, L. C., Wang, S. T., & Arrellaga, J., (1998), "Computer Program APILE Plus – A Program for the Analysis of the Axial Capacity of Driven Piles" ENSOFT, INC., Austin, TX.

10. US Army Corps of Engineers (1999), *Reconnaissance Report – Kansas Citys, Missouri and Kansas Flood Damage Reduction Project,* Kansas City District.

11. Wolff, T. F., (1985), "Analysis and Design of Embankment Dam Slopes: A Probabilistic Approach", Doctoral Dissertation presented to Purdue University, West Lafayette, IND.

12. Wright, S. G., (1999), "UTEXAS 4 – A Computer Program for Slope Stability Calculations", prepared for the Department of the Army, U. S. Army Corps of Engineers, Washington D. C.

13. Corps of Engineers Engineering Manuals, Technical Letters, Etc as referenced within.

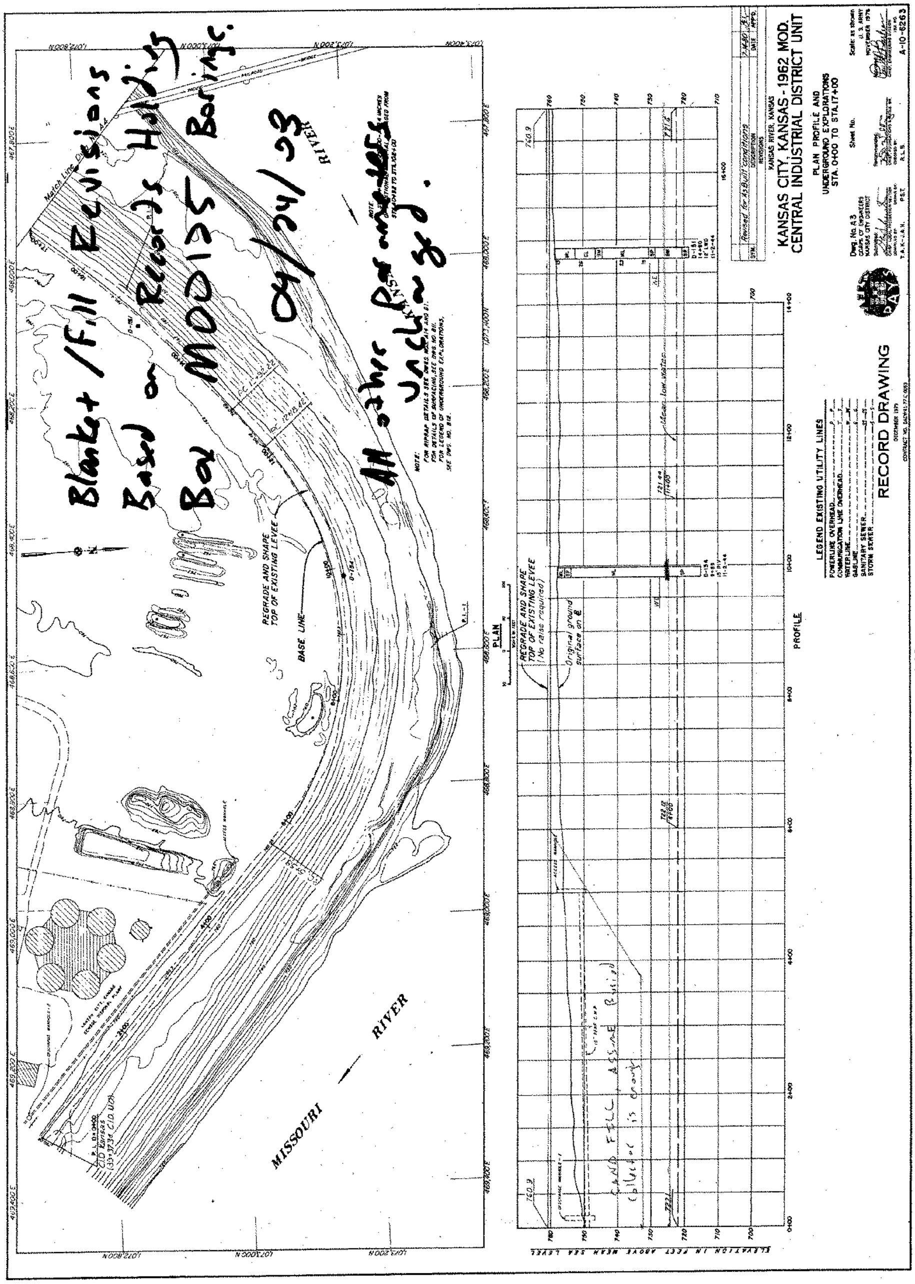
14. US Army Corps of Engineers (April 1973), *Design Memorandum No 4 – Central Industrial District Kansas Unit*, Kansas City District.

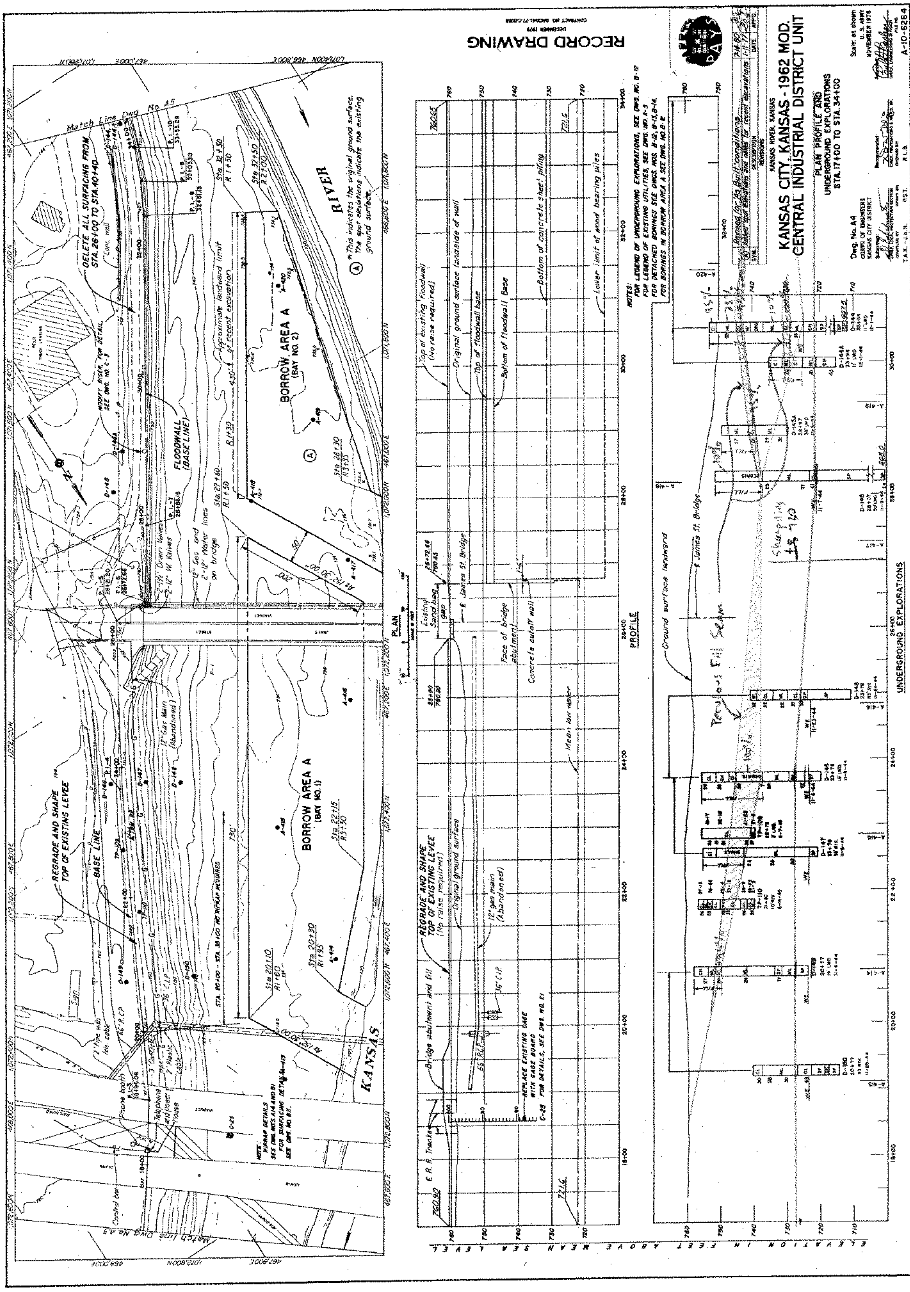
A-4.11 SUPPLEMENTAL EXHIBITS

EXHIBIT A-4.1

CID-KS Levee – Existing Conditions Underseepage Analysis

CID-KS Levee Kansas City Phase 2 Feasibility
Friday, July 13, 2007 Revised: Thursday, April 24, 2008
Levee Foundation Information - EXISTING CONDITIONS, Water to Top of Levee


Blanket /Fill Revisions Based on Records Holding Borings MOD13S 09/24/03
All other Parameters Unchanged.
MISSOURI RIVER
REGRADE AND SHAPE TOP OF EXISTING LEVEE
BASE LINE
PLAN
PROFILE
LEGEND EXISTING UTILITY LINES
RECORD DRAWING
KANSAS CITY, KANSAS - 1962 MOD.
CENTRAL INDUSTRIAL DISTRICT UNIT
PLAN PROFILE AND UNDERGROUND EXPLORATIONS
STA. 0+00 TO STA. 17+00
A-10-6263


RECORD DRAWING
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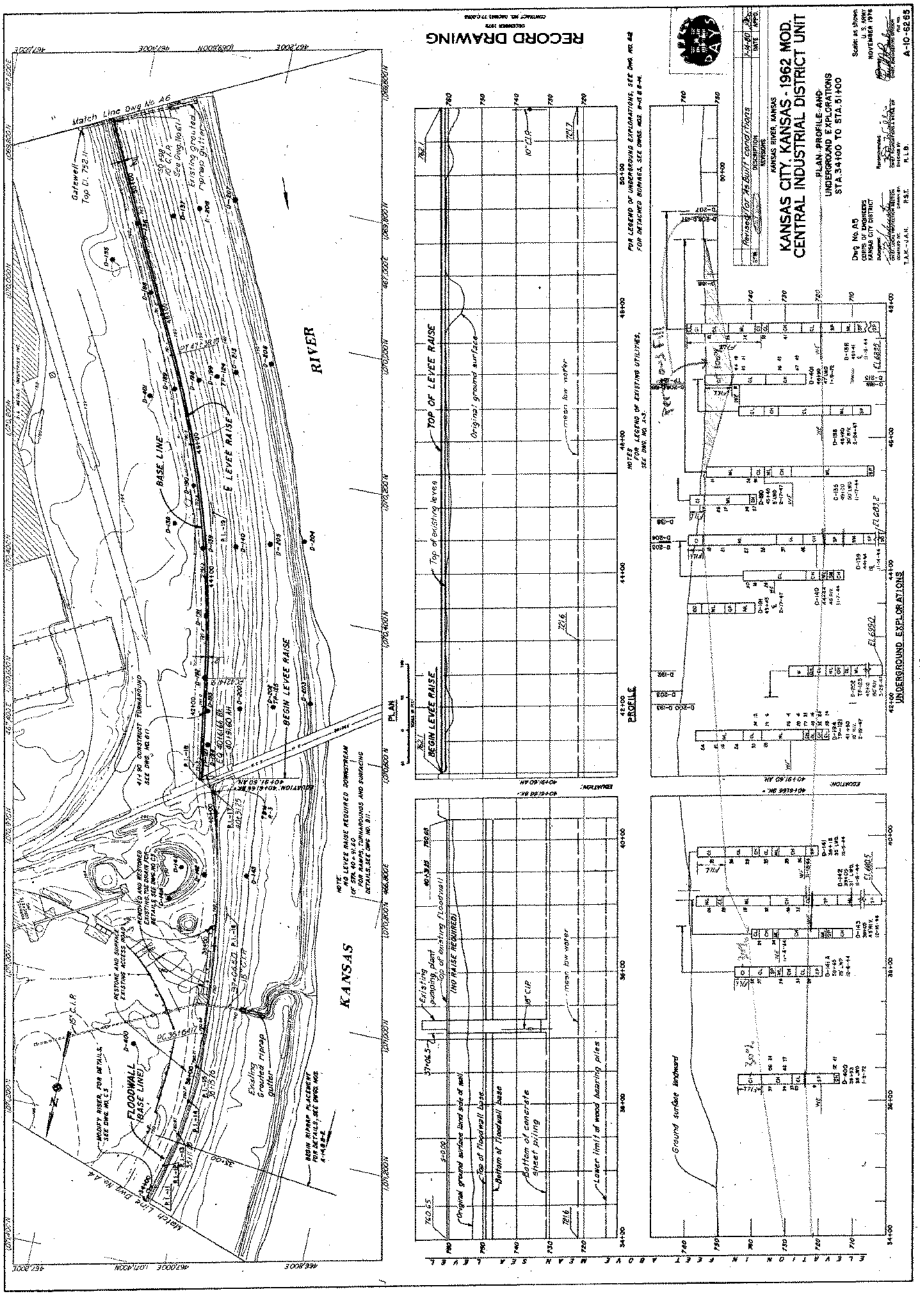
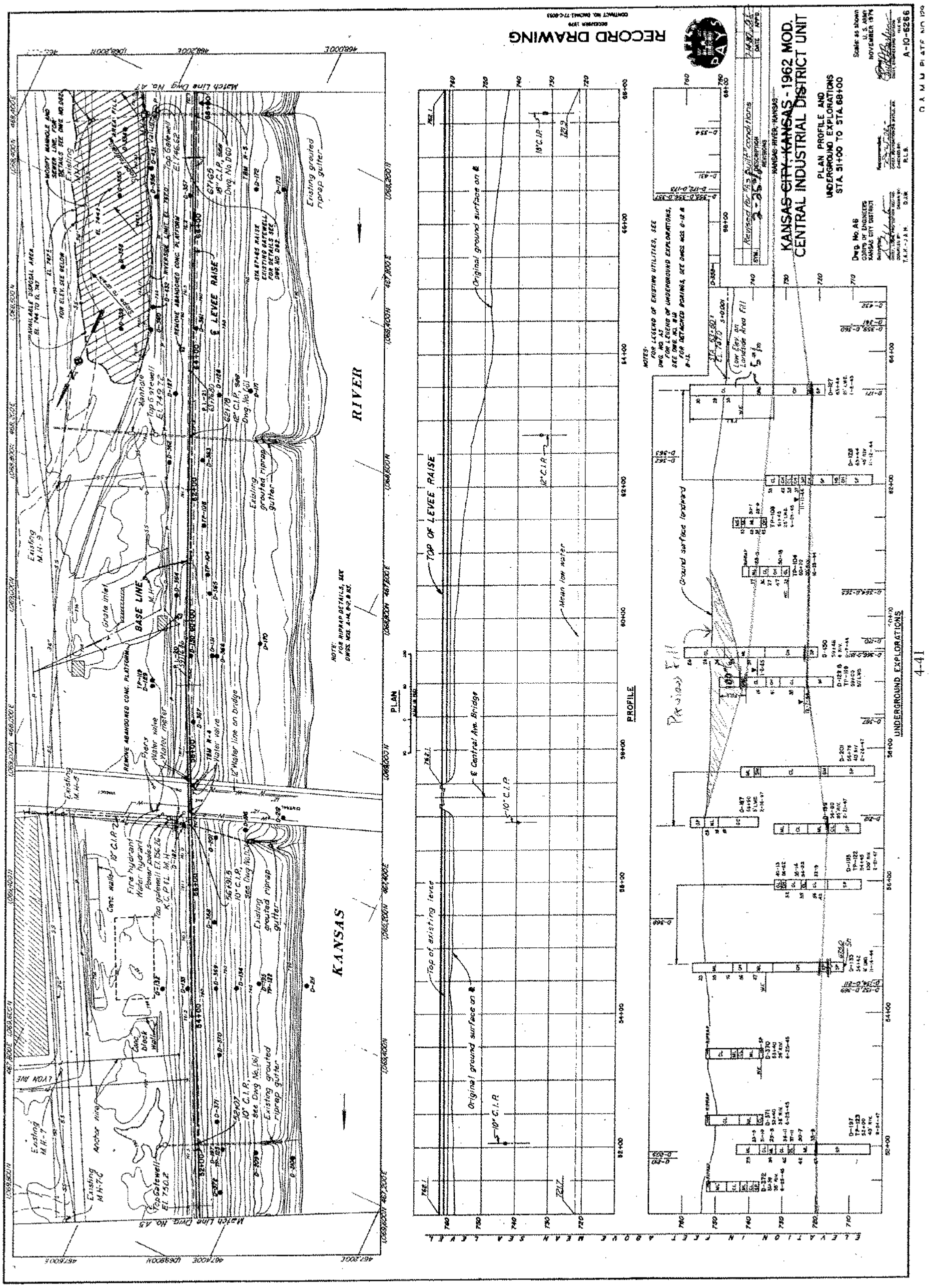
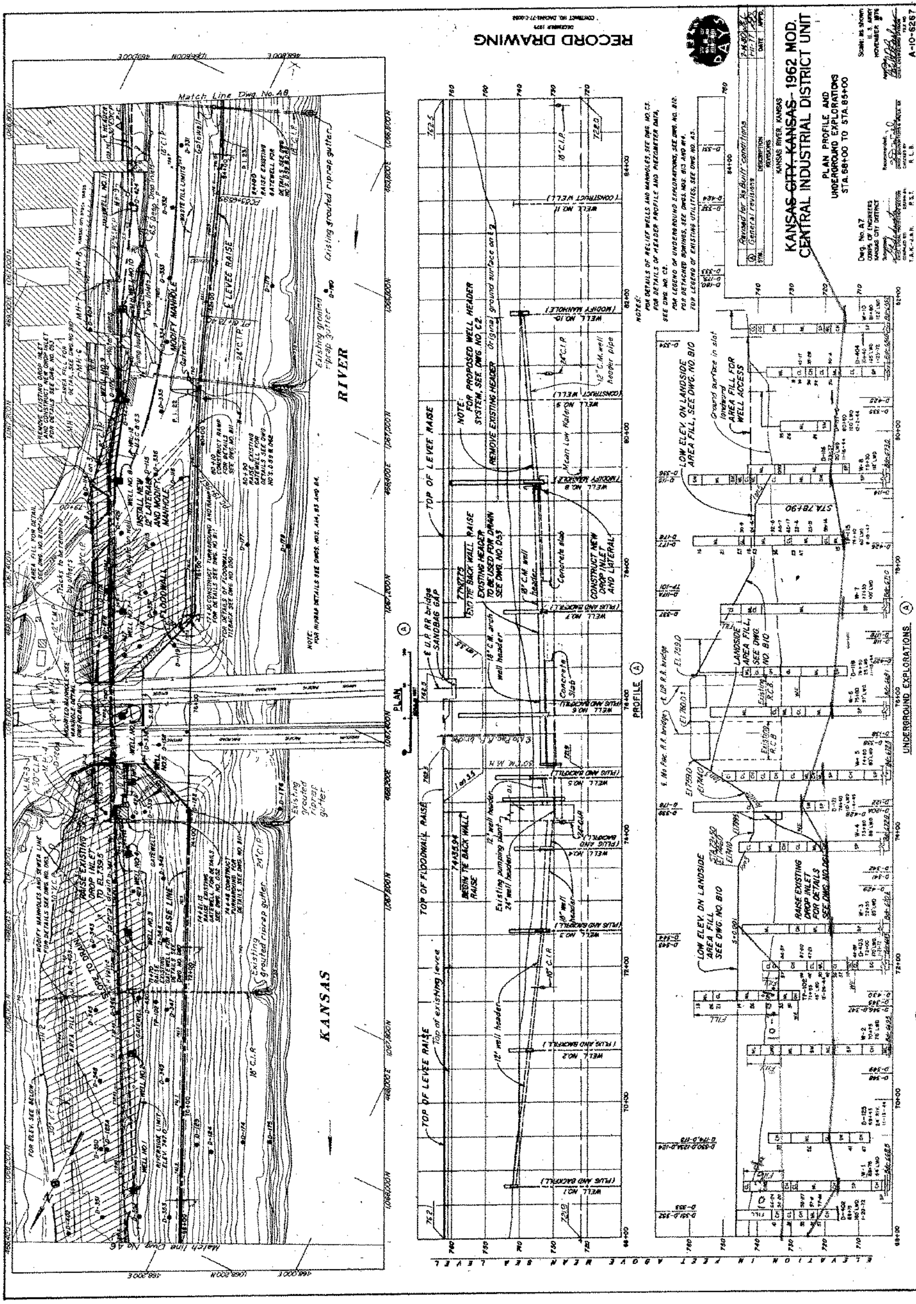
CENTRAL INDUSTRIAL DISTRICT UNIT
PLAN PROFILE AND UNDERGROUND EXPLORATIONS
STA. 17+00 TO STA. 34+00
BORROW AREA A (BAY NO. 2)
BORROW AREA A (BAY NO. 1)
FLOODWALL (BASE LINE)
KANSAS RIVER
PLAN
PROFILE
UNDERGROUND EXPLORATIONS
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
A-10-6254


RECORD DRAWING
KANSAS
RIVER
PLAN
PROFILE
UNDERGROUND EXPLORATIONS
BEGIN LEVEE RAISE
TOP OF LEVEE RAISE
FLOODWALL (BASE LINE)
BASE LINE
KANSAS CITY, KANSAS - 1962 MOD.
CENTRAL INDUSTRIAL DISTRICT UNIT
PLAN-PROFILE-AND-UNDERGROUND EXPLORATIONS
STA.34+00 TO STA.51+00
Dwg No. A5
A-10-6265
ELEVATION IN FEET ABOVE MEAN SEA LEVEL


RECORD DRAWING
KANSAS
RIVER
PLAN
PROFILE
UNDERGROUND EXPLORATIONS
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
KANSAS CITY, KANSAS - 1962 MOD.
CENTRAL INDUSTRIAL DISTRICT UNIT
PLAN PROFILE AND
UNDERGROUND EXPLORATIONS
STA. 51+00 TO STA. 68+00
4-41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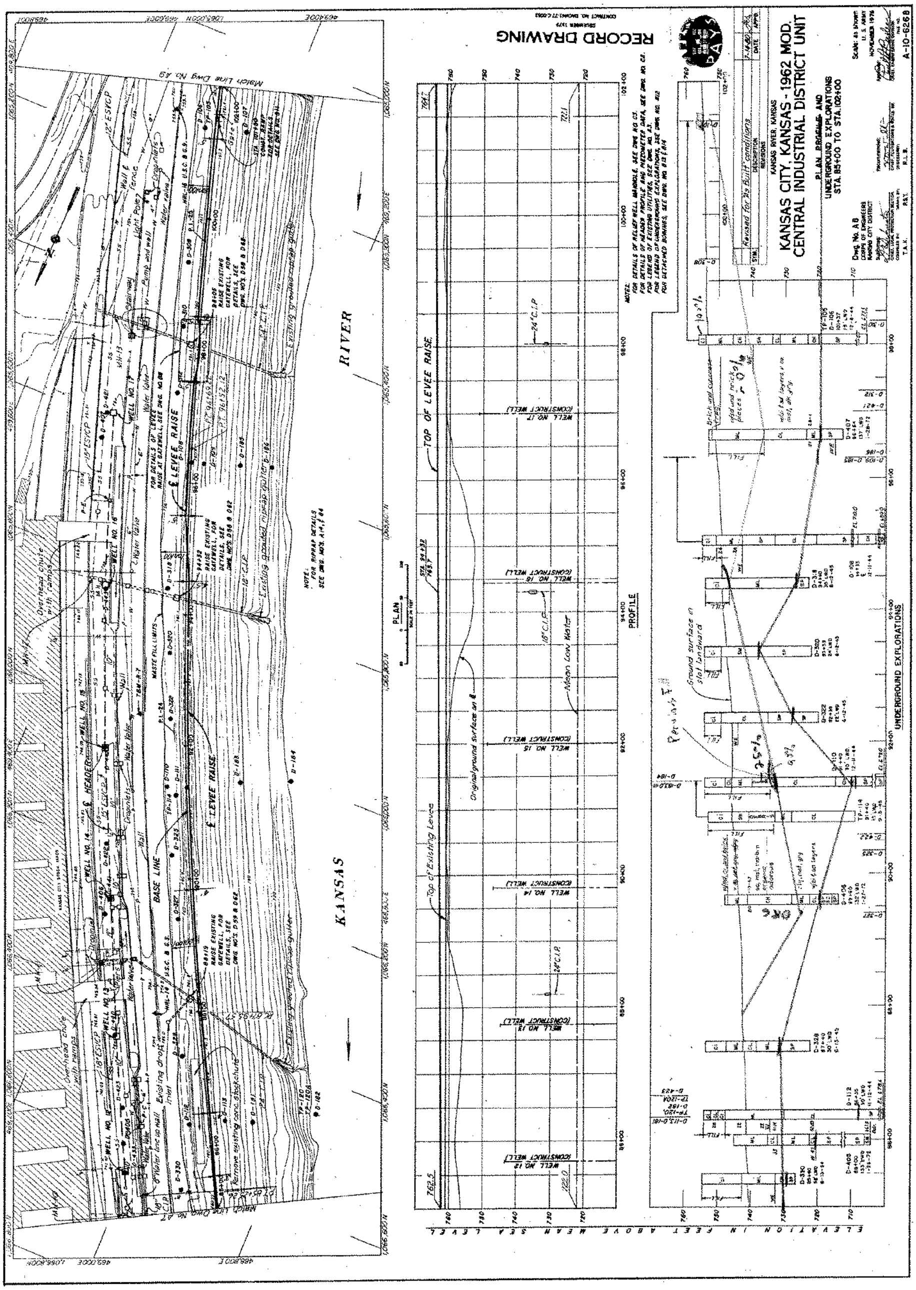
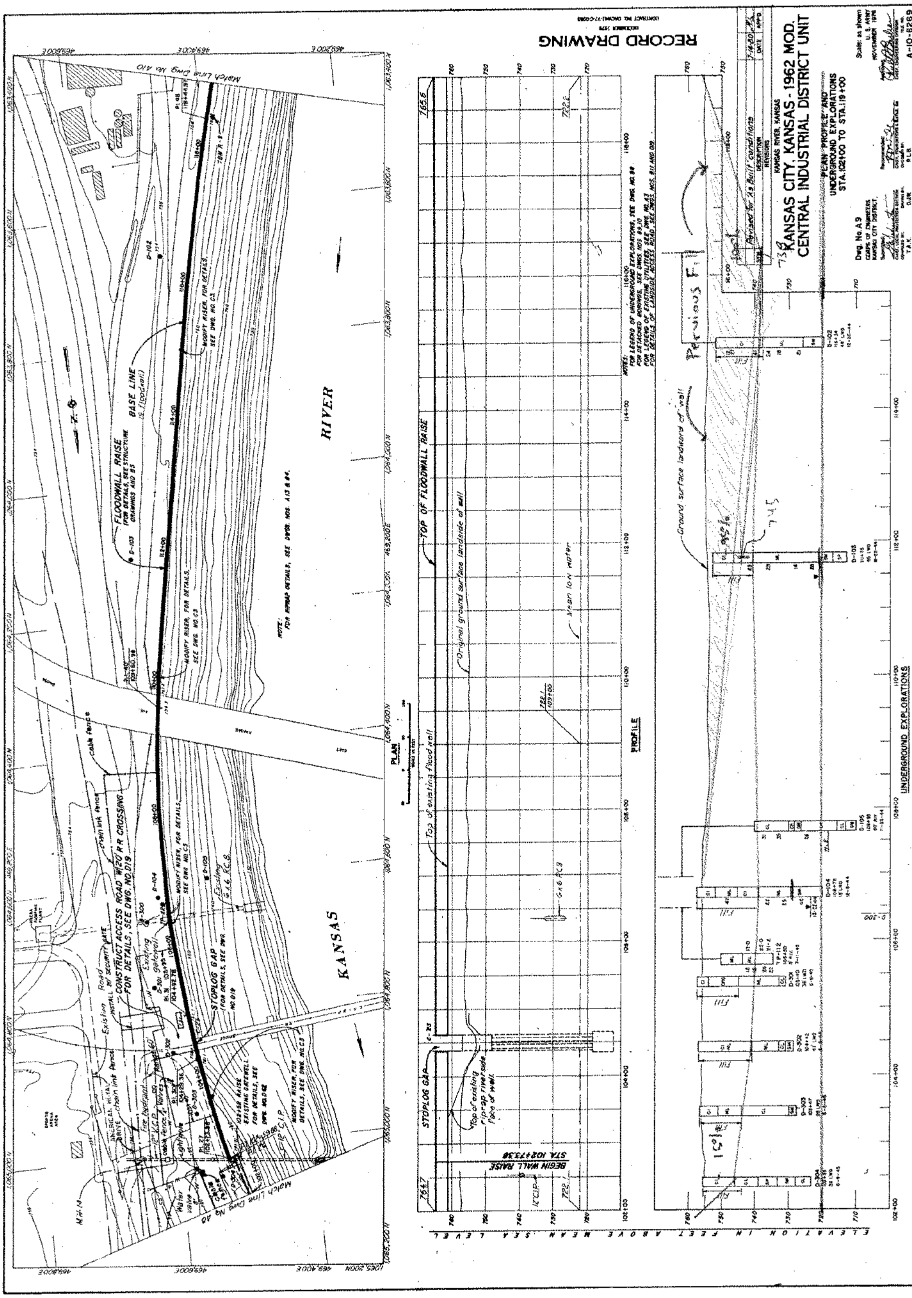
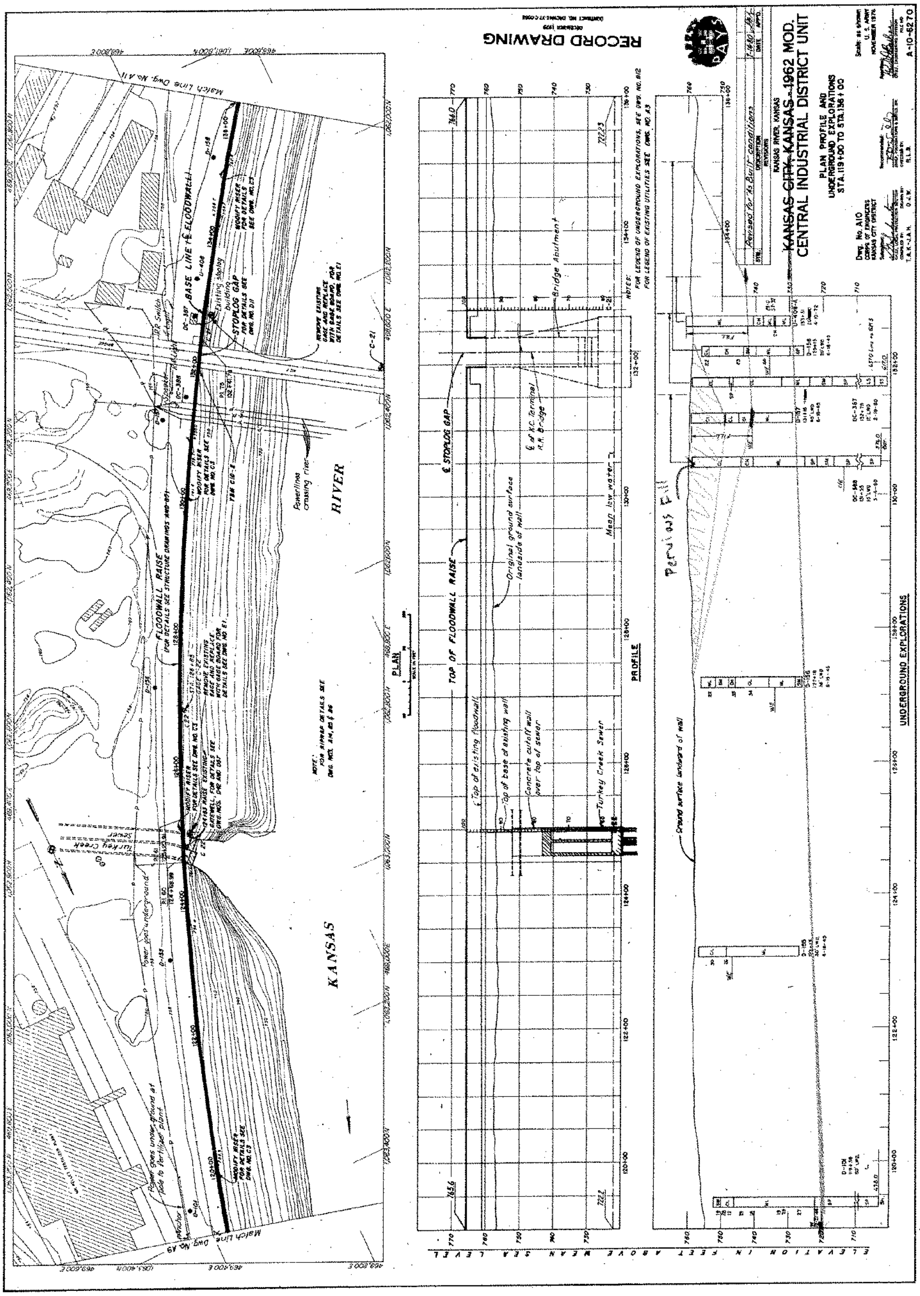
CENTRAL INDUSTRIAL DISTRICT UNIT
PLAN
PROFILE
UNDERGROUND EXPLORATIONS
KANSAS
RIVER
ELEVATION IN FEET ABOVE MEAN SEA LEVEL

4-42


RECORD DRAWING
KANSAS RIVER
PLAN
PROFILE
UNDERGROUND EXPLORATIONS
TOP OF LEVEE RAISE
KANSAS CITY, KANSAS - 1962 MOD.
CENTRAL INDUSTRIAL DISTRICT UNIT
A-10-6268
O. & M. M. PLATE NO. 131



RECORD DRAWING
KANSAS CITY, KANSAS - 1962 MOD.
CENTRAL INDUSTRIAL DISTRICT UNIT
PLAN, PROFILE AND UNDERGROUND EXPLORATIONS
STA. 102+00 TO STA. 119+00
A-10-6269
KANSAS
RIVER
FLOODWALL RAISE
STOPLOG GAP
TOP OF FLOODWALL RAISE
PLAN
PROFILE
UNDERGROUND EXPLORATIONS
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
4-44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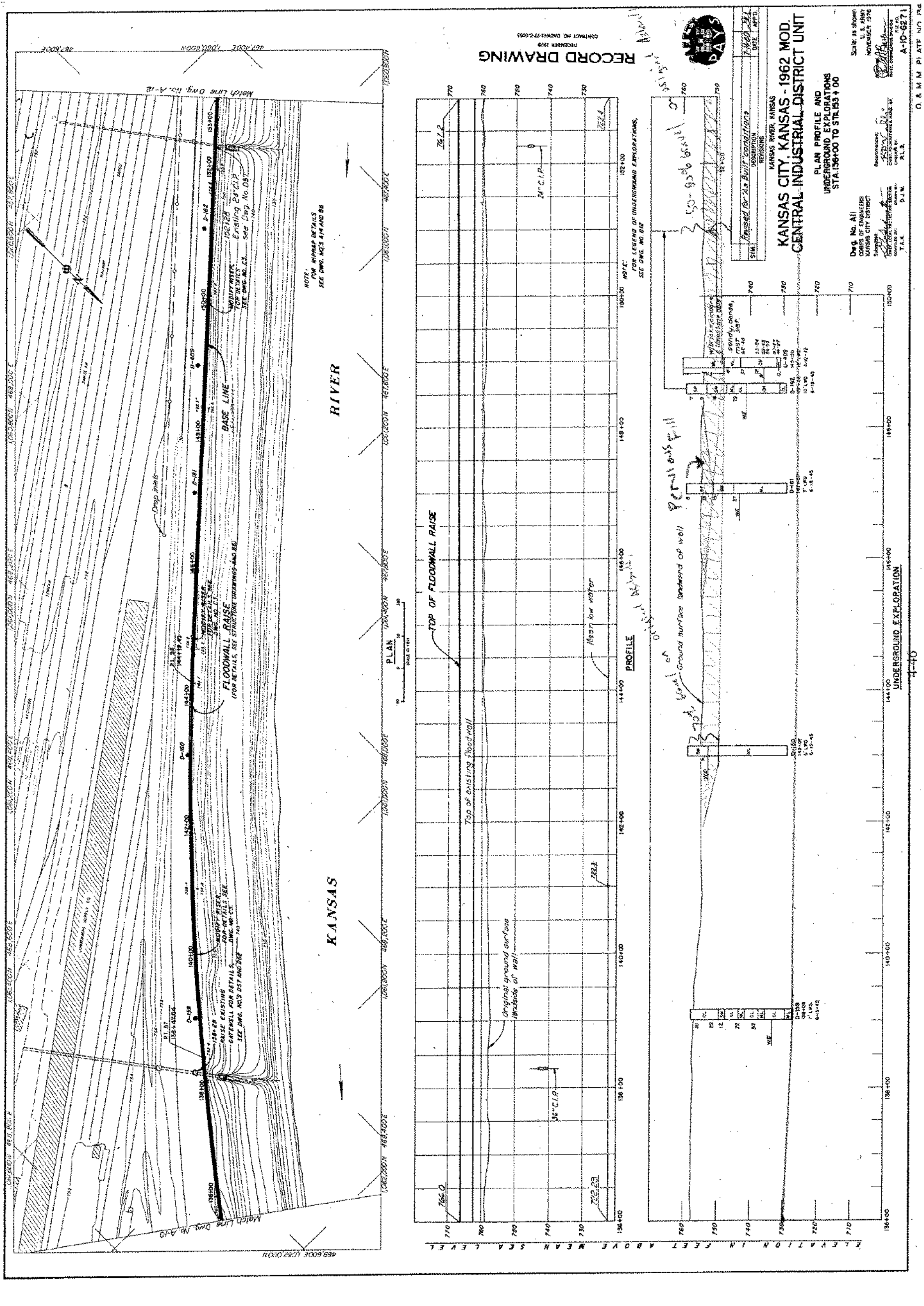
CENTRAL INDUSTRIAL DISTRICT UNIT
PLAN PROFILE AND UNDERGROUND EXPLORATIONS
STA. 119+00 TO STA. 136+00
Dwg. No. A10
A-10-6270
O. & M.M. PLATE NO. 133
PLAN
PROFILE
UNDERGROUND EXPLORATIONS
KANSAS
RIVER
TOP OF FLOODWALL RAISE
STOPLOG GAP
FLOODWALL RAISE
Turkey Creek Sewer
Mean low water
ELEVATION IN FEET ABOVE MEAN SEA LEVEL


RECORD DRAWING
KANSAS
RIVER
FLOODWALL RAISE
BASE LINE
PLAN
TOP OF FLOODWALL RAISE
PROFILE
UNDERGROUND EXPLORATION
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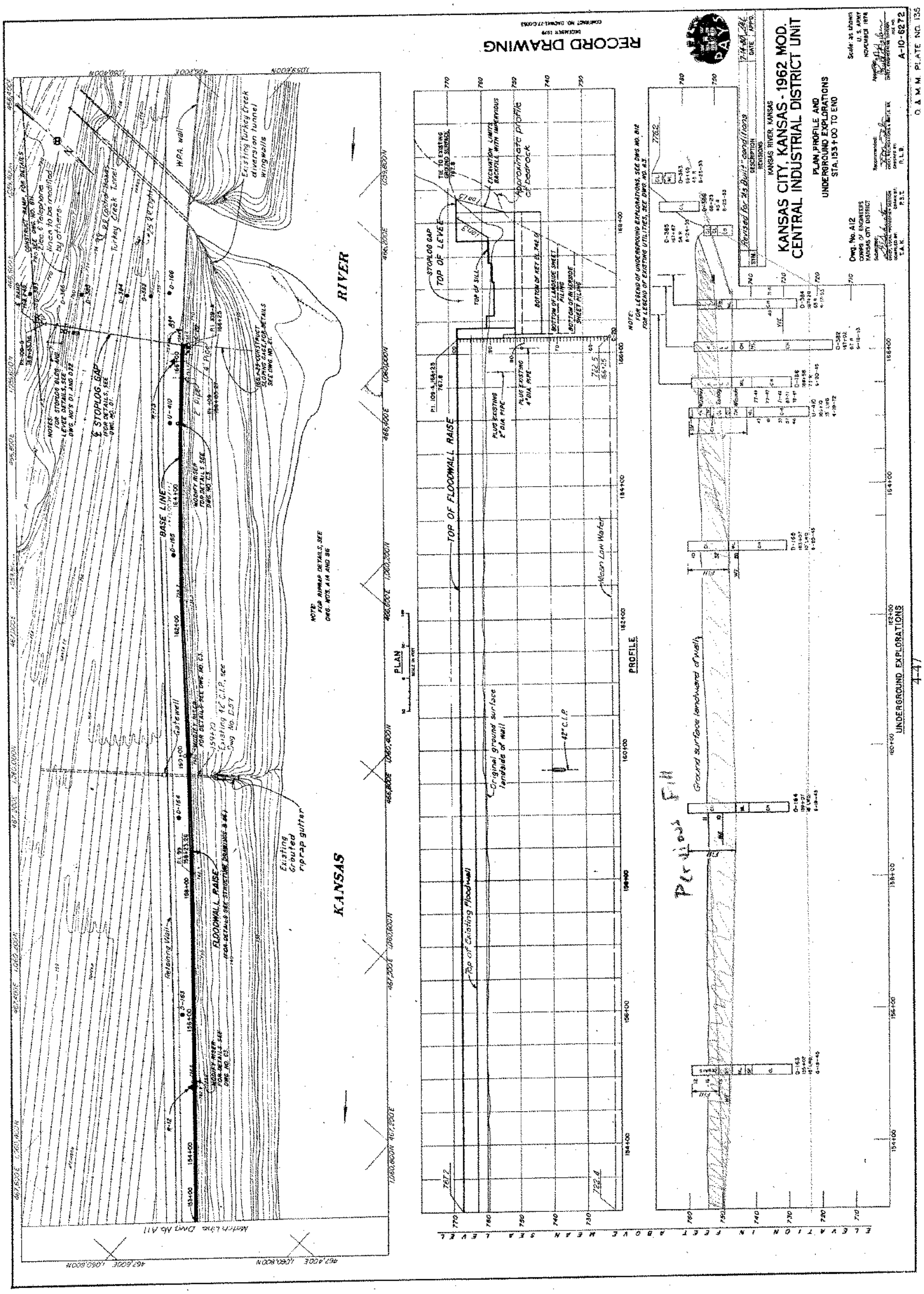
CENTRAL INDUSTRIAL DISTRICT UNIT
PLAN PROFILE AND
UNDERGROUND EXPLORATIONS
STA.136+00 TO STA.153+00
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
A-10-62(71)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KANSAS CITY, KANSAS - 1962 MOD.
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PLAN, PROFILE AND
UNDERGROUND EXPLORATIONS
STA. 153+00 TO END
KANSAS RIVER, KANSAS
A-10-6272
O. & M. PLATE NO. 135
KANSAS
RIVER
PLAN
PROFILE
UNDERGROUND EXPLORATIONS
BASE LINE
FLOODWALL RAISE
STOPLOG GAP
TOP OF LEVEE
TOP OF FLOODWALL RAISE
ELEVATION IN FEET ABOVE MEAN SEA LEVEL

EXHIBIT A-4.2
Typical shape of the normal probability distribution function showing the expected value or mean, E[X]

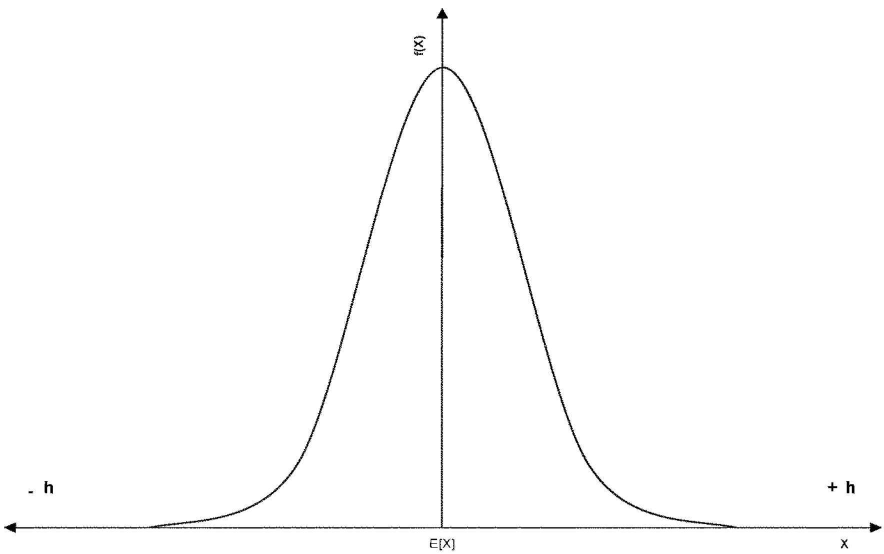


EXHIBIT A-4.3
Typical shape of the log-normal distribution function showing the expected value, E[X]

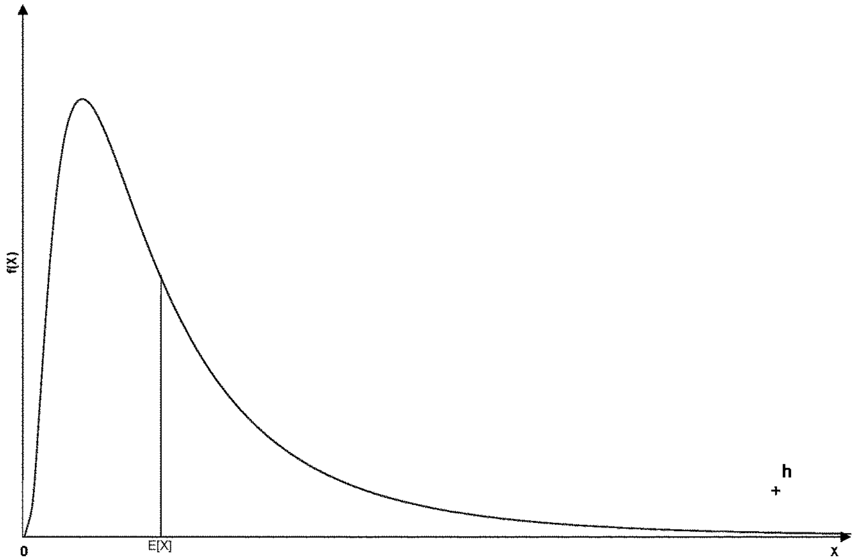


EXHIBIT A-4.4
Hypothetical normal probability distribution showing the probabilistic parameters

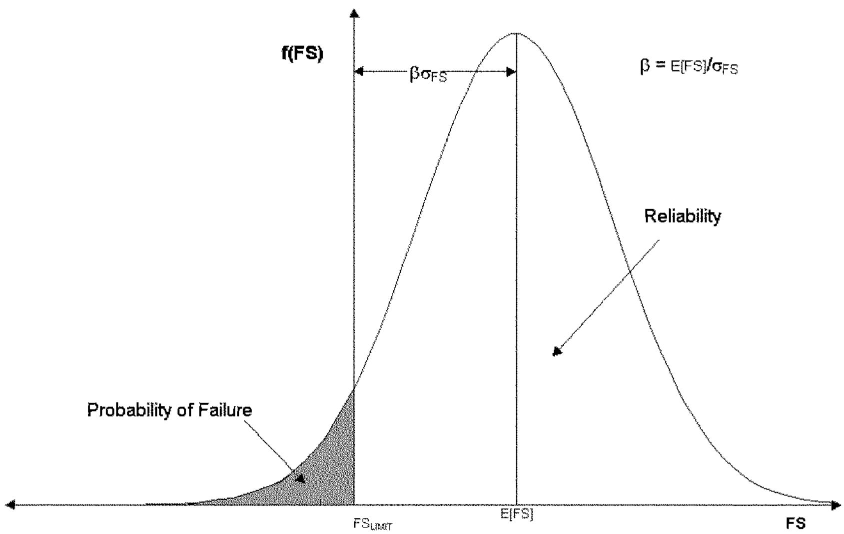


EXHIBIT A-4.5
Normal probability distribution for the natural log of the factor of safety, assuming that the factor of safety is log-normally distributed

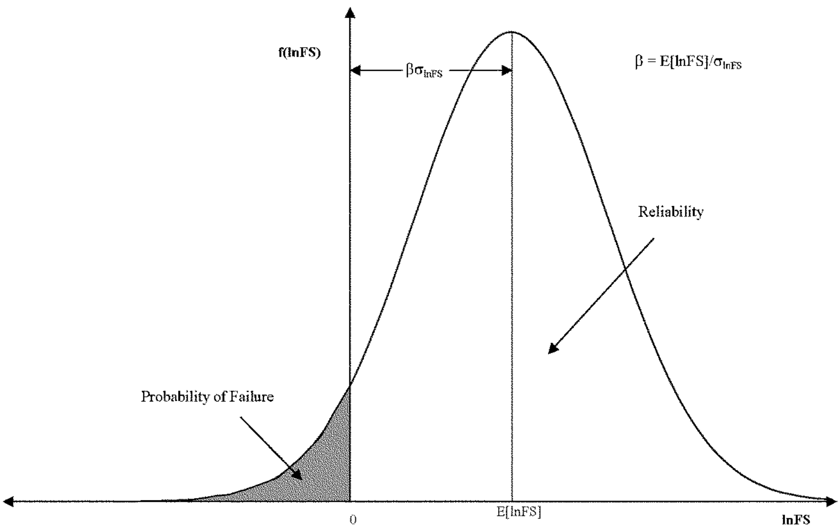


EXHIBIT A-4.6
Normal probability distribution for the natural log of the hydraulic gradient, assuming that the hydraulic gradient is log-normally distributed where the failure gradient is defining the limit state

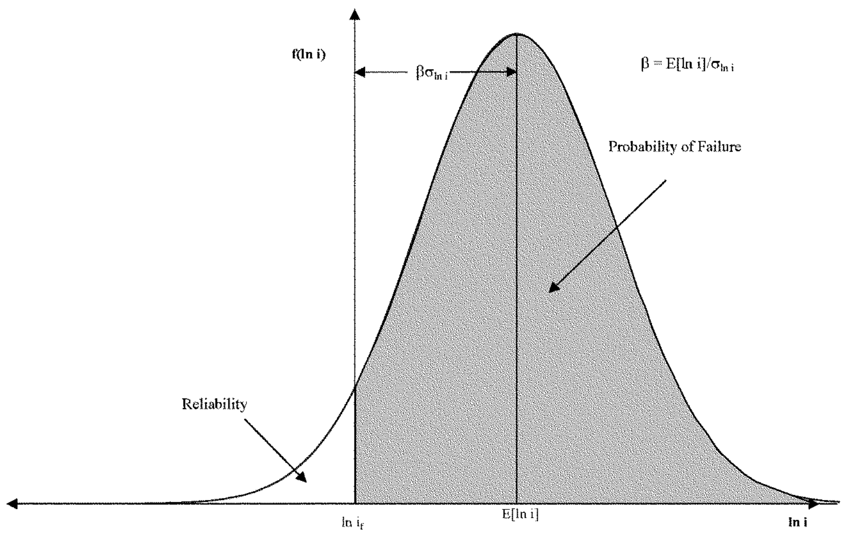


EXHIBIT A-4.7
The probability distribution curve illustrating the assumptions used in developing the Taylor Series Approximation

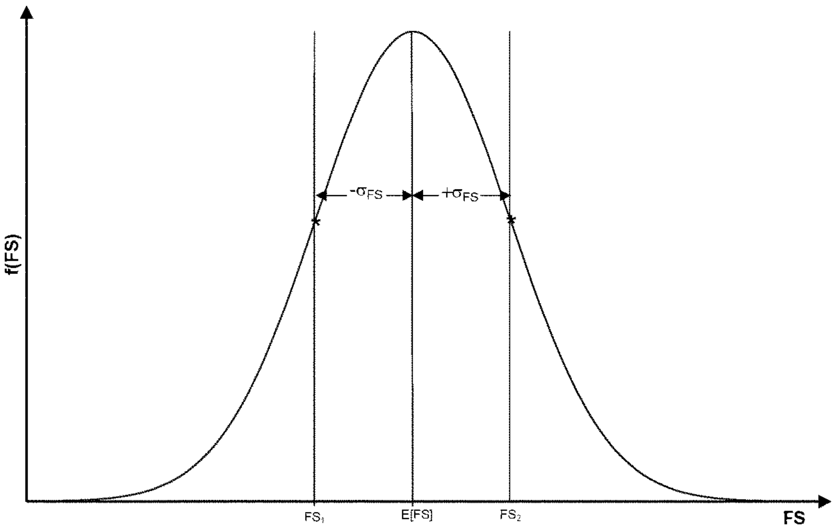


Exhibit A-4.8: Underseepage Probability of Failure vs. Factor of Safety

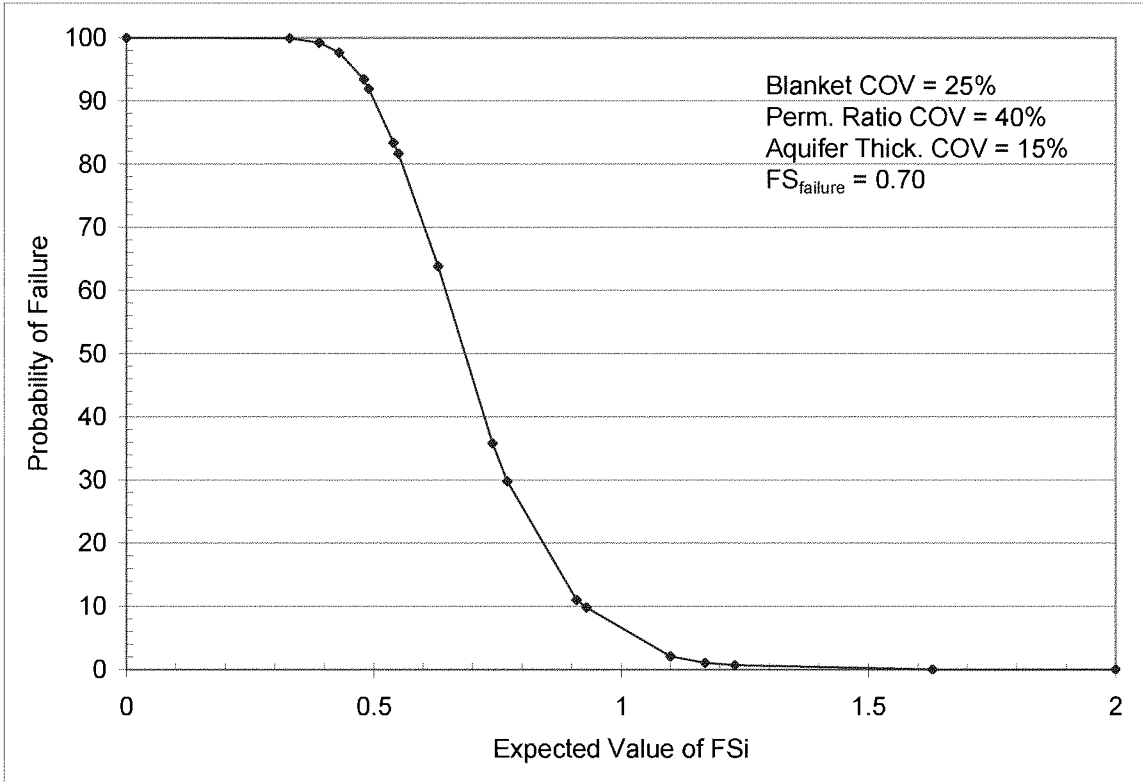


Exhibit A-4.9: Underseepage Probability of Failure at Station 85+00 (Stockyard Area)

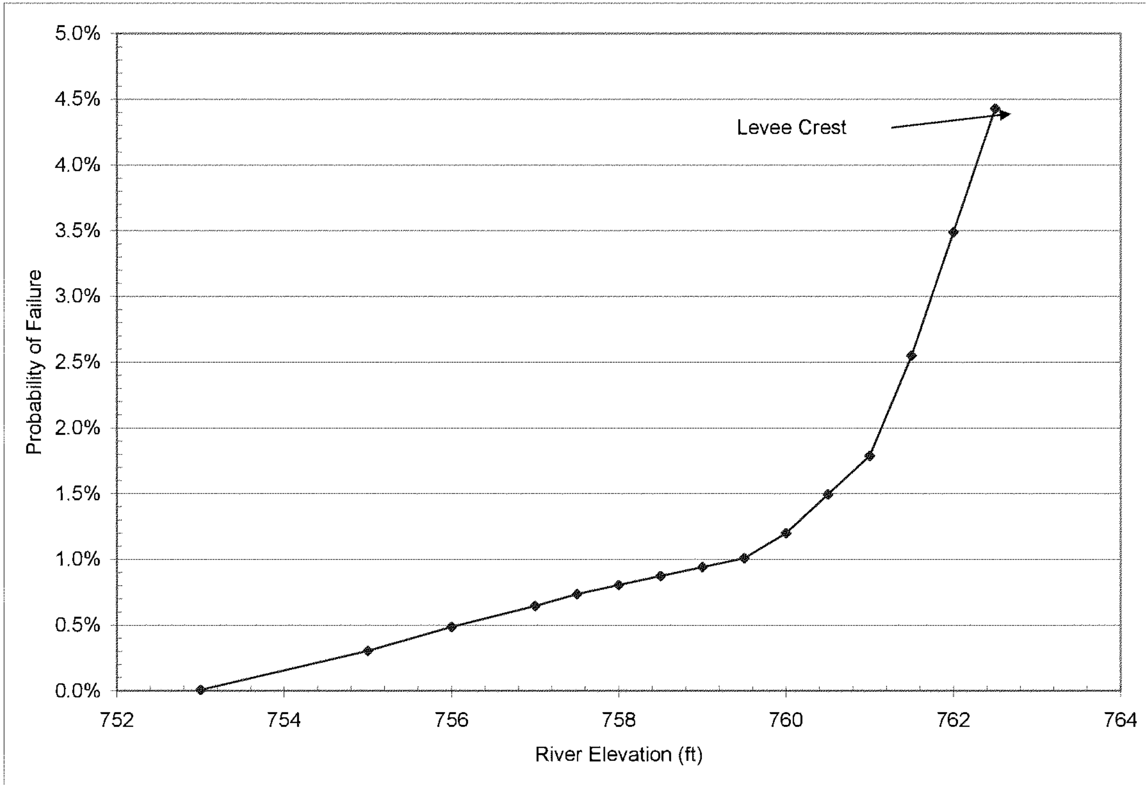


Exhibit A-4.10

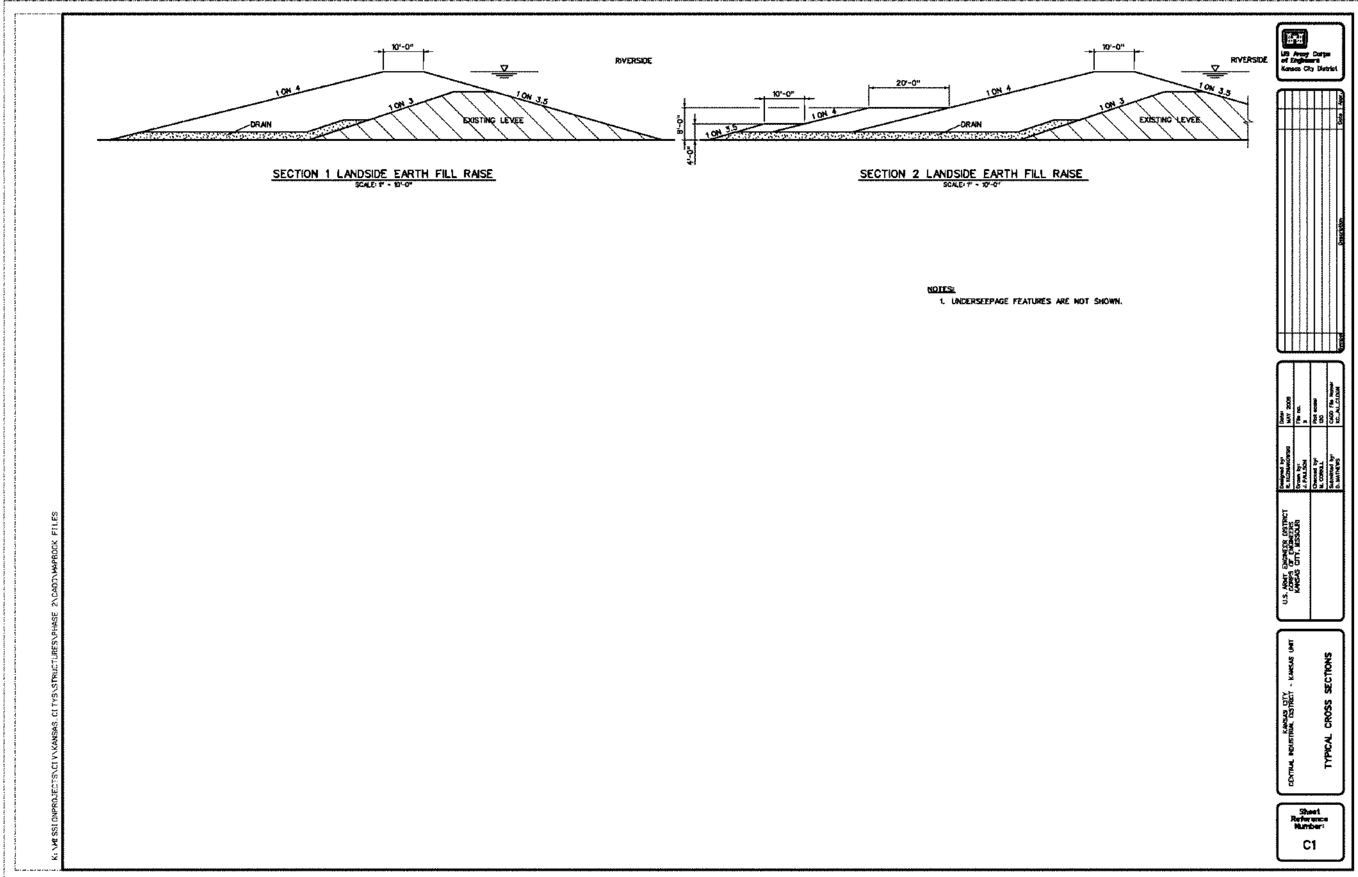


EXHIBIT A-4.11

Proposed Levee Raise Summary

1/2

CID Kansas 7 Jun 2008
RSK

Levee Raises
For n500+3; Compare Raise to Armourdale Sections.
Maintain Riverside 1V on 3.5H

1) Sta 83+01.39 (State Line) to Sta 18+15± (RR Bridge)

No Raise → Resurface Road

2) Sta 19+73±(RR Bridge) to Sta 25+90 (James Street)

Minimal Raise 0.2' to 0.8' (Total Ht 1.1' to 4.8')
✓(Use Section 1 from Armourdale)

3) Sta 40+91.6 (RR Bridge) to Sta 57+07± (Central Ave)
· Blanket = 30'; h_o = 11.1'
Raise 0.4' to 1.6' (Total Ht 11' to 13.7')
✓(Use Section 1 from Armourdale)

4) Sta 57+49± (Central Avenue) to 74+35.95 (RR Bridge)
· Blanket thickness = 30'; h_o = 12.4'
Raise 1.8' to 2.9' (Total Ht 13.7' to 14.4' w/ area fill)
(Use Section 1 from Armourdale)

5) Sta 77+27.75 (RR Bridge) to 102+73.38 (Floodwall)
· Blanket = 30'; h_o = 15'
Raise 2.85' to 3.6' (Total Ht 12.5' to 17.9')
(Use Section 2 from Armourdale)

From Armourdale
Section 1 - maybe up to 17' (Analyzed for 16.5')
Section 2 - above 17' (Analyzed for 20.1')

2/2

7 JUN 2008
RSK

SECTION 1 STA 90+00
- H = 16.5'
- BLANKET THICKNESS = 25' $\phi' = 26°$
- h_o = 13.6'

FINAL SECTION: 1V on 4H LANDSIDE SLOPE FS = 1.5

SECTION 2 STA 245+50
- H = 20.1'
- BLANKET THICKNESS = 18' $\phi' = 26°$
- h_o = 11.6'
- STABILITY BERM 1: 8' THICK; 20' BENCH, 1V on 4H
BERM 2: 4' THICK; 10' BENCH, 1V on 3.5H

Station	Existing Levee Crest Elevation (ft)	Existing Landside Elevation (ft)	Existing Levee Landside Height (ft)	Existing Landside Slope (cot)	n500+3 Levee Crest Elevation (ft)	n500+3 Raise (ft)	Proposed Section Number	First Berm: Berm Height (ft)	First Berm: Berm Width (ft)	Second Berm: Berm Height	Second Berm: Berm Width	Proposed Landside Height (ft)	Distance from Exist. Centerline to Exist. Toe (ft)	Distance from Exist. Centerline to Proposed n500+3 Toe (ft)	Remarks
83+01.29 to 85+00	760.90	753.0	7.90	4.0	760.90	0.00	1					7.90	37	37	No Raise
85+00 to 89+37.34	760.90	753.0	7.90	4.0	760.90	0.00	1					7.90	37	37	No Raise
Sta Equation Change															
0+00 to 5+00	760.90	760.9	0.00	4.0	760.90	0.00	1					0.00	5	5	No Raise
5+00 to 10+00	760.90	760.9	0.00	4.0	760.90	0.00	1					0.00	5	5	No Raise
10+00 to 15+00	760.90	760.9	0.00	4.0	760.90	0.00	1					0.00	5	5	No Raise
15+00 To 18+15	760.90	760.9	0.00	4.0	760.90	0.00	1					0.00	5	5	No Raise
Railroad Bridge															
19+73 to 20+00	760.90	757.0	3.90	4.0	761.10	0.20	1					4.10	21	22	
20+00 to 25+00	760.90	757.0	3.90	4.0	761.60	0.70	1					4.60	21	26	
25+00 to 25+90	760.90	757.0	3.90	4.0	761.70	0.80	1					4.70	21	27	
James Street Bridge															
26+72.66 to 40+31.25	760.65						Floodwall								
40+31.25 to 45+00	762.10	752.0	10.10	4.0	762.50	0.40	~~1~~ 3					10.50	45	48	
45+00 to 50+00	762.10	752.0	10.10	4.0	763.00	0.90	~~1~~ 3					11.00	45	52	
50+00 to 55+00	762.10	752.0	10.10	4.0	763.40	1.30	~~1~~ 3					11.40	45	55	
55+00 to 57+07	762.10	752.0	10.10	4.0	763.60	1.50	1					11.60	45	57	
Central Avenue Bridge															
57+49 to 60+00	762.10	750.0	12.10	3.5	763.90	1.80	1					13.90	47	67	
60+00 to 63+00	762.10	750.0	12.10	3.5	764.20	2.10	1					14.20	47	69	
63+00 to 65+00	762.10	**751.0**	11.10	3.5	764.40	2.30	1					13.40	44	67	Area Fill to Elevation 751
65+00 to 70+00	762.20	**751.0**	11.20	3.5	764.90	2.70	1					13.90	44	70	Area Fill to Elevation 751
70+00 to 74+35.94	762.50	**751.0**	11.50	3.5	765.30	2.80	1					14.30	45	72	Area Fill to Elevation 751
Railroad Bridge															
77+27.75 to 80+00	762.50	**749.0**	13.50	3.5	765.80	3.30	2	8	20	4	10	16.80	52	114	Area Fill to Elevation 749
80+00 to 85+00	762.50	**749.0**	13.50	3.0	766.10	3.60	2	8	20	4	10	17.10	46	116	Area Fill to Elevation 749
85+00 to 90+00	762.80	**749.0**	13.80	3.0	766.50	3.70	2	8	20	4	10	17.50	46	118	Area Fill to Elevation 749
90+00 to 95+00	763.50	**749.0**	14.50	3.0	766.90	3.40	2	8	20	4	10	17.90	49	119	Area Fill to Elevation 749
95+00 to 100+00	764.10	750.0	14.10	3.0	767.30	3.20	2	8	20	4	10	17.30	47	115	
100+00 to 102+73.39	764.55	760.0	4.55	3.0	767.45	2.90	1					7.45	19	45	
102+73.39 to 166+25						0.00	Floodwall								
168+16 to 168+36	767.80	759.0	8.80	3.0	771.60	3.80	1					12.60	31	69	

Notes:
1. Numbers shown indicate minimum real estate needs.
2. Existing levee crest elevations are average for the given reach, n500+3 crest elevations are maximum for the reach.
3. All n500+3 landside slopes are considered to be 1V on 4H for simplicity, even though the second berm is 1V on 3.5H.
4. Existing Landside Elevations in bold are the proposed area fill elevations, and not the actual existing elevations.

EXHIBIT A-4.12

T-Wall on Levee Section

2 July 2008

RSK

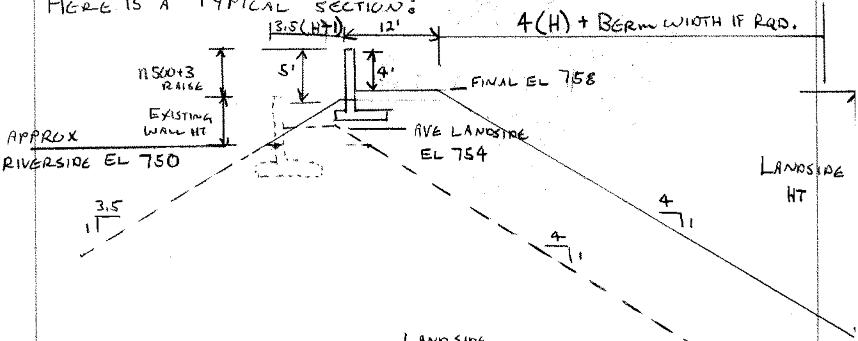
1/2

CIA Kansas

- Review of locations where a floodwall could be replaced with a T-wall on levee

The only possible location that is feasible appears to be between James Street Bridge and the abandoned RR track, sta 26+70± to 40+30±

Here is a typical section:



Height of levee raise = (Existing landside wall ht) + (n500+3 raise) − 4'

- The average existing landside wall height = 7.5'

- The average n500+3 raise = 1'

So; levee raise = 7.5' + 1' − 4' = 4.5'

* Plus add 4' for ave elevation difference between riverside and land side of floodwall

The new toe of the levee from the existing floodwall:

3.5(4.5' − 1 + 4') + 12' + [4(Landside HT) + Berm width if Rqd)

≅ 40' + [4(Landside HT) + Berm width if Rqd.)

2 JULY 2008
RSK
2/2

SEE ATTACHED INFO FOR HEIGHT DATA

STA	LANDSIDE HEIGHT (FT) n500+3	BERM WIDTH (FT)	LEVEE TOE (FT)
26+73 TO 30+00	8'	—	72'
30+00 TO 35+00	10'	—	80'
35+00 TO 38+50	10' (FILL TO 748)	—	80'
38+50 TO 40+32	4'	—	56'

* BASED ON ATTACHED MAP: IT APPEARS THIS MAY BE A FEASIBLE ALTERNATIVE, ALTHOUGH IT DOES COME FAIRLY CLOSE TO THE BUILDING.

* WOULD NEED ABOUT A 40' PERMANENT EASEMENT.

THE ONLY OTHER WALL IS FROM STA 102+73 TO 166+25

• THE EXISTING WALL HAS A 9' TO 12' EXPOSED FLOODWALL STEM AND THERE WILL BE A 2.5' TO 4.0' n500+3 RAISE

* BY INSPECTION THIS REACH IS NOT FEASIBLE FOR A T-WALL ON LEVEE SECTION.

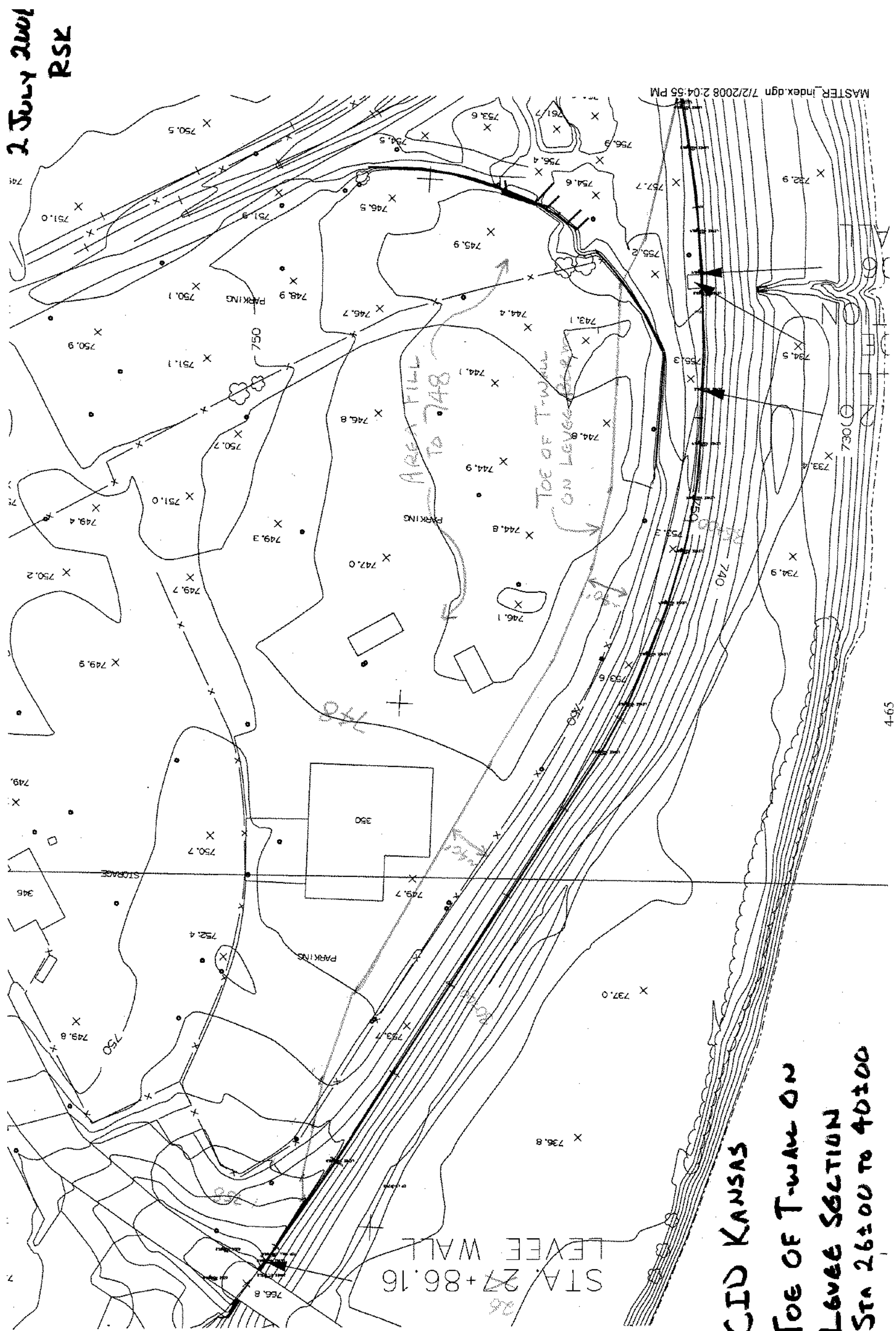
Kansas City Levees - Phase 2 Feasibility Study
CID Kansas
Levee Raise Data - started 2 July 2007, revised 2 July 2008

Station	Existing Levee Crest Elevation (ft)	Existing Levee Landside Elevation (ft)	Existing Levee Landside Height (ft)	Proposed n500+3 Elevation (ft)	Proposed Levee Landside Height (ft)	Proposed Levee Landside Raise (ft)	Remarks
83+01.29 to 85+00	760.9	753.0	7.9	760.90	7.90	0.00	
85+00 to 89+37.34*	760.9	753.0	7.9	760.90	7.90	0.00	
0+00 to 5+00	760.9	760.9	0.0	760.90	0.00	0.00	
5+00 to 10+00	760.9	760.9	0.0	760.90	0.00	0.00	
10+00 to 15+00	760.9	760.9	0.0	760.90	0.00	0.00	
15+00 to 18+15	760.9	760.9	0.0	760.90	0.00	0.00	
18+15 to 19+73	N/A						High Bridge
19+73 to 20+00	760.9	757.0	3.9	761.10	4.10	0.20	
20+00 to 25+00	760.9	757.0	3.9	761.60	4.60	0.70	
25+00 to 25+90	760.9	757.0	3.9	761.70	4.70	0.80	
25+90 to 26+72.66	N/A						Sandbag Gap
26+72.66 to 30+00	760.9	750.0	10.9	761.80	11.80	0.90	Floodwall
30+00 to 35+00	760.9	748.0	12.9	761.90	13.90	1.00	Floodwall
35+00 to 38+50	760.9	744.0	16.9	762.00	18.00	1.10	Floodwall
38+50 to 40+31.25	760.9	754.0	6.9	762.10	8.10	1.20	Floodwall
40+31.25 to 45+00	762.1	752.0	10.1	762.50	10.50	0.40	
45+00 to 50+00	762.1	752.0	10.1	763.00	11.00	0.90	
50+00 to 55+00	762.1	752.0	10.1	763.40	11.40	1.30	
55+00 to 57+07	762.1	752.0	10.1	763.60	11.60	1.50	
57+07 to 57+49	N/A				0.00		High Ground
57+49 to 60+00	762.1	750.0	12.1	763.90	13.90	1.80	
60+00 to 63+00	762.1	750.0	12.1	764.20	14.20	2.10	
63+00 to 65+00	762.1	**751.0**	11.1	764.40	13.40	2.30	
65+00 to 70+00	762.2	**751.0**	11.2	764.90	13.90	2.70	
70+00 to 74+35.94	762.5	**751.0**	11.5	765.30	14.30	2.80	
74+35.94 to 77+27.75	N/A						Sandbag Gap
77+27.75 to 80+00	762.5	**749.0**	13.5	765.80	16.80	3.30	Slot
80+00 to 85+00	762.5	**749.0**	13.5	766.10	17.10	3.60	Slot
85+00 to 90+00	762.8	**749.0**	13.8	766.50	17.50	3.70	Slot
90+00 to 95+00	763.5	**749.0**	14.5	766.90	17.90	3.40	Slot
95+00 to 100+00	764.1	750.0	14.1	767.30	17.30	3.20	
100+00 to 102+73.38	764.6	760.0	4.5	767.45	7.45	2.90	
102+73.38 to 104+40	765.0	755.0	10.0	767.65	12.65	2.65	Floodwall
104+40 to 104+60	N/A						Stoplog Gap
104+60 to 105+00	765.1	755.0	10.1	767.70	12.70	2.60	Floodwall
105+00 to 110+00	765.3	749.0	16.3	768.00	19.00	2.70	Floodwall
110+00 to 115+00	765.4	753.0	12.4	768.40	15.40	3.00	Floodwall
115+00 to 120+00	765.6	754.0	11.6	768.80	14.80	3.20	Floodwall
120+00 to 125+00	765.7	755.0	10.7	769.20	14.20	3.50	Floodwall
125+00 to 130+00	765.9	756.0	9.9	769.50	13.50	3.60	Floodwall
130+00 to 135+00	766.0	758.0	8.0	769.90	11.90	3.90	Floodwall
135+00 to 140+00	766.3	752.0	14.3	770.20	18.20	3.90	Floodwall
140+00 to 145+00	766.6	752.0	14.6	770.50	18.50	3.90	Floodwall
145+00 to 150+00	767.0	752.0	15.0	770.70	18.70	3.70	Floodwall
150+00 to 155+00	767.3	752.0	15.3	771.00	19.00	3.70	Floodwall
155+00 to 160+00	767.5	752.0	15.5	771.30	19.30	3.80	Floodwall
160+00 to 166+25	767.8	754.0	13.8	771.60	17.60	3.80	Floodwall
166+25 to 168+10	N/A						Stoplog Gap
168+10 to 168+33	767.8	754.0	13.8	771.63	17.63	3.83	

*Station Equation Changes?
Bold landside ground surface elevations indicate area fill elevation required for n500+3 raise.

7.5' TALL FLOODWALL EXPOSED STEM
AVG 1.0' n500+3 RAISE

9' TO 12' TALL FLOODWALL EXPOSED STEM
~~AVG~~ 2.5' TO 4.0' n500+3 RAISE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LEVEE WALL
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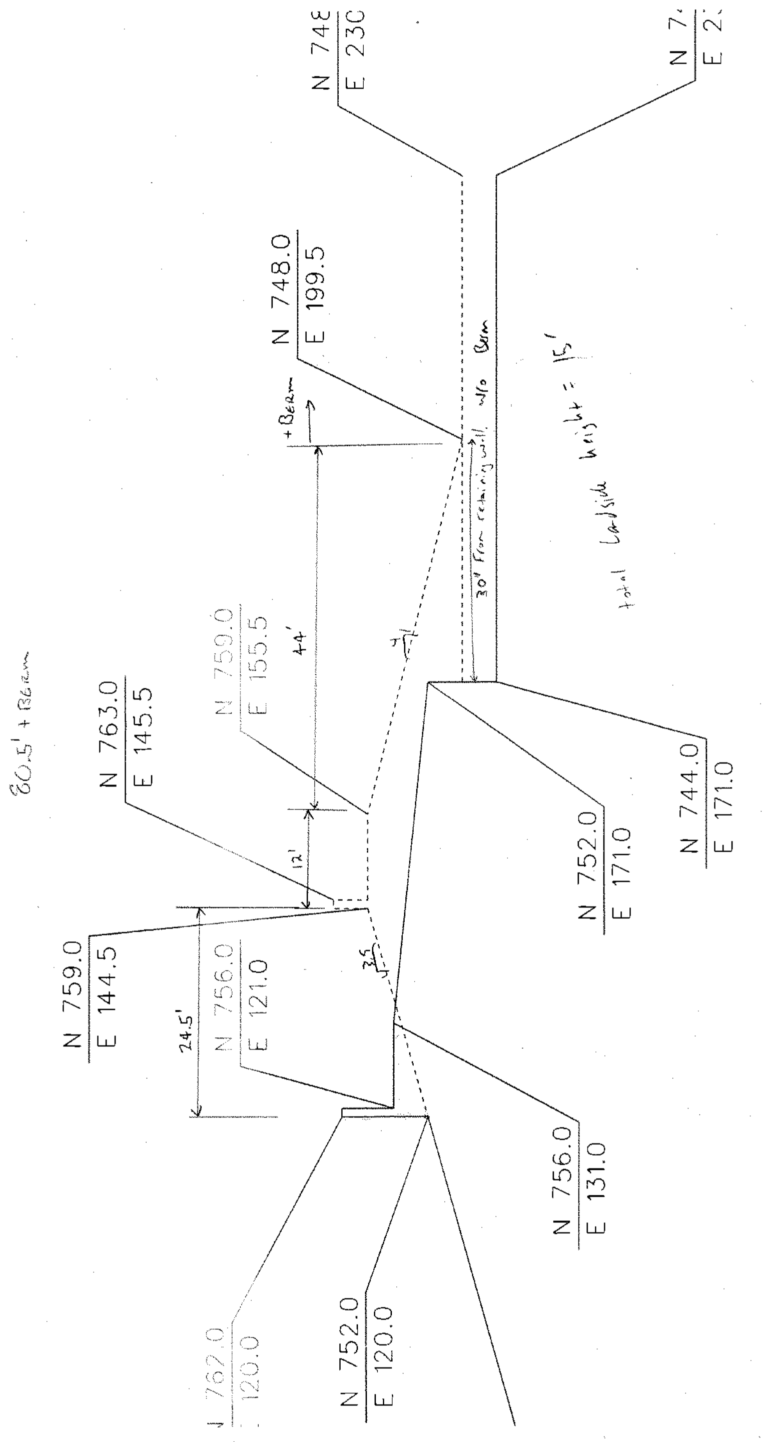
CID Kansas
Toe of T-wall on
Levee Section
Sta 26+00 to 40+00



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CiD-KS Levee Kansas City Phase 2 Feasibility
Thursday, April 24, 2008 Revised: Friday, April 25, 2008
Levee Foundation Information - 500+3 CONDITIONS, Water to Top of Levee

Exhibit A-4.13
Underseepage Analysis without Relief Wells

Exhibit A-4.14
Area Fill Design - Station 32+00 to 38+00

IMPERVIOUS AREA FILL DESIGN

CID-KS Levee Kansas City Phase 2 Feasibility
Monday, April 28, 2008 Revised:
Levee Foundation Information - 500+3, Water to Top of Levee, Station 28+00 top 38+00 Impervious Area Fill for Underseepage Control

Blanket Unit Weight = 115.0 pcf (saturated)
Rubble Fill Unit Weight = 115.0 pcf (saturated)

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Blanket Elevation (msl)	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio River Side k_f/k_{bR}	Permeability Ratio Land Side k_f/k_{bL}	Impervious Blanket Thickness (ft) River Side z_{bR}	Impervious Blanket Thickness (ft) Levee z_{bB}	Impervious Blanket Thickness (ft) Land Side z_{bL}	Pervious Blanket Thickness (ft) d	Seepage Length (ft) River Side L_1	Seepage Length (ft) Levee L_2	Seepage Length (ft) Land Side L_3	Factor c_1	Factor c	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_0	Computed Hydraulic Gradient i_0	Critical Hydraulic Gradient i_c	Factor of Safety for Piping (i_c/i_0)	Remarks
28+00 to 31+00	762.0	740	750	721	721	750	670	12.0	300	300	19	24	29	51	90	65	1000	0.001856	0.001501	89	666	9.74	0.34	0.84	2.51	
31+00 to 32+50	762.0	740	748	721	721	748	670	14.0	300	300	19	23	27	51	90	65	1000	0.001855	0.001556	89	643	11.29	0.42	0.84	2.02	
32+50 to 34+00	762.0	740	**748**	721	721	748	670	14.0	300	300	19	23	27	51	45	65	1000	0.001855	0.001556	45	643	11.96	0.44	0.84	1.90	
34+00 to 35+00	762.0	740	**748**	721	721	748	670	14.0	800	800	19	23	27	51	0	65	1000	0.001136	0.000953	0	1050	13.18	0.49	0.84	1.73	
35+00 to 36+50	762.0	740	**748**	721	721	748	670	14.0	800	800	19	23	27	51	0	65	1000	0.001136	0.000953	0	1050	13.18	0.49	0.84	1.73	toe at retaining wall
36+50 to 37+00	762.0	740	**748**	721	721	748	670	14.0	800	800	19	23	27	51	0	85	1000	0.001136	0.000953	0	1050	12.95	0.48	0.84	1.76	toe at retaining wall
36+50 to 37+00	762.0	740	755	721	721	755	670	7.0	800	800	19	26.5	34	51	0	50	1000	0.001136	0.000849	0	1178	6.71	0.20	0.84	4.27	check toe 25 ft from centerline
37+00 to 37+50	762.0	740	**748**	721	721	748	670	14.0	800	800	19	23	27	51	0	125	1000	0.001136	0.000953	0	1050	12.51	0.46	0.84	1.82	toe at retaining wall
37+00 to 37+50	762.0	740	755	721	721	755	670	7.0	800	800	19	26.5	34	51	0	70	1000	0.001136	0.000849	0	1178	6.61	0.19	0.84	4.34	check toe 40 ft from centerline
37+50 to 38+00	762.0	740	**748**	721	721	748	670	14.0	800	800	19	23	27	51	0	200	1000	0.001136	0.000953	0	1050	11.76	0.44	0.84	1.94	toe at retaining wall
37+50 to 38+00	765.0	740	755	721	721	755	670	10.0	800	800	19	26.5	34	51	0	80	1000	0.001136	0.000849	0	1178	9.36	0.28	0.84	3.06	check toe 50 ft from centerline

PERVIOUS AREA FILL DESIGN

CID-KS Levee Kansas City Phase 2 Feasibility
Monday, April 28, 2008 Revised:
Levee Foundation Information - 500+3, Water to Top of Levee, Station 28+00 top 38+00 Pervious Area Fill for Underseepage Control

Blanket Unit Weight = 115.0 pcf (saturated)
Rubble Fill Unit Weight = 115.0 pcf (saturated)

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Blanket Elevation (msl)	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio River Side k_f/k_{bR}	Permeability Ratio Land Side k_f/k_{bL}	Impervious Blanket Thickness (ft) River Side z_{bR}	Impervious Blanket Thickness (ft) Levee z_{bB}	Impervious Blanket Thickness (ft) Land Side z_{bL}	Pervious Blanket Thickness (ft) d	Seepage Length (ft) River Side L_1	Seepage Length (ft) Levee L_2	Seepage Length (ft) Land Side L_3	Factor c_1	Factor c	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_0	Computed Hydraulic Gradient i_0	Critical Hydraulic Gradient i_c	Factor of Safety for Piping (i_c/i_0)	Remarks
28+00 to 31+00	762.0	740	750	721	721	750	670	12.0	300	300	19	24	29	51	90	65	1000	0.001855	0.001501	89	666	9.74	0.34	0.84	2.51	
31+00 to 32+50	762.0	740	748	721	721	748	670	14.0	300	300	19	23	27	51	90	65	1000	0.001855	0.001556	89	643	11.29	0.42	0.84	2.02	
32+50 to 34+00	762.0	740	**749**	721	721	746	670	13.0	300	300	19	22	25	51	45	65	1000	0.001855	0.001617	45	618	11.04	0.44	0.84	1.91	
34+00 to 35+00	762.0	740	**749**	721	721	745	670	13.0	800	800	19	21.5	24	51	0	65	1000	0.001136	0.001011	0	990	12.20	0.51	0.84	1.66	
35+00 to 36+50	762.0	740	**749**	721	721	744	670	13.0	800	800	19	21	23	51	0	65	1000	0.001136	0.001032	0	989	12.18	0.53	0.84	1.59	toe at retaining wall
36+50 to 37+00	762.0	740	**749**	721	721	745	670	13.0	800	800	19	21.5	24	51	0	85	1000	0.001136	0.001011	0	990	11.97	0.50	0.84	1.69	toe at retaining wall
36+50 to 37+00	762.0	740	755	721	721	755	670	7.0	800	800	19	26.5	34	51	0	50	1000	0.001136	0.000849	0	1178	6.71	0.20	0.84	4.27	check toe 25 ft from centerline
37+00 to 37+50	762.0	740	**749**	721	721	746	670	13.0	800	800	19	22	25	51	0	125	1000	0.001136	0.000990	0	1010	11.57	0.46	0.84	1.82	toe at retaining wall
37+00 to 37+50	762.0	740	755	721	721	755	670	7.0	800	800	19	26.5	34	51	0	70	1000	0.001136	0.000849	0	1178	6.61	0.19	0.84	4.34	check toe 40 ft from centerline
37+50 to 38+00	762.0	740	**749**	721	721	747	670	13.0	800	800	19	22.5	26	51	0	200	1000	0.001136	0.000971	0	1030	10.89	0.42	0.84	2.01	toe at retaining wall
37+50 to 38+00	765.0	740	755	721	721	755	670	10.0	800	800	19	26.5	34	51	0	80	1000	0.001136	0.000849	0	1178	9.36	0.28	0.84	3.06	check toe 50 ft from centerline

EXHIBIT A-4.15

Relief Well Design – Station 32+00 to 38+00

Project Name

UNDERSEEPAGE ANALYSIS WITHOUT WELLS

Levee Foundation Information - Case Analyzed

Blanket Unit Weight = 115.0 pcf (saturated)

N500+3 Relief Well System Summary - Station 32+00 to 38+00

Well	Distance From Seepage Entrance (ft)	Station	Discharge Elevation (ft)	0.8*Q_w (cfs)	Q_w (cfs)
1	65	33+50	747	1.03	1.28
2	65	34+00	747	0.93	1.17
3	65	34+50	746	0.94	1.17
4	65	35+00	746	0.85	1.06
5	65	35+25	745	0.86	1.08
6	65	35+50	745	0.86	1.08
7	65	36+00	745	0.82	1.02
8	65	36+25	745	0.77	0.96
9	75	36+37	745	0.72	0.90
10	75	36+50	745	0.74	0.93
11	85	36+75	746	0.74	0.93
12	100	37+00	746	0.80	1.00
			Total	**10.06**	**12.58**

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	51	ft
h_o =	9.74	ft
TOL =	762	ft elevation
Landside =	750.0	ft elevation
Bottom Blanket =	721	ft elevation
blanket =	29.0	ft
z, Landside Toe =	155	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	11.59	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
1	65	3350	747.0	0.94	0.30	1.00
2	65	3400	747.0	0.85	0.27	1.00
3	65	3450	746.0	0.86	0.27	1.00
4	65	3500	746.0	0.78	0.25	1.00
5	65	3525	745.0	0.80	0.25	1.00
6	65	3550	745.0	0.80	0.25	1.00
7	65	3600	745.0	0.76	0.24	1.00
8	65	3625	745.0	0.71	0.23	1.00
9	75	3637	745.0	0.67	0.21	1.00
10	75	3650	745.0	0.69	0.22	1.00
11	85	3675	746.0	0.68	0.22	1.00
12	100	3700	746.0	0.73	0.23	1.00
13				0.00	0.00	0.00
14				0.00	0.00	0.00
15				0.00	0.00	0.00
16				0.00	0.00	0.00
17				0.00	0.00	0.00
18				0.00	0.00	0.00
19				0.00	0.00	0.00
20				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

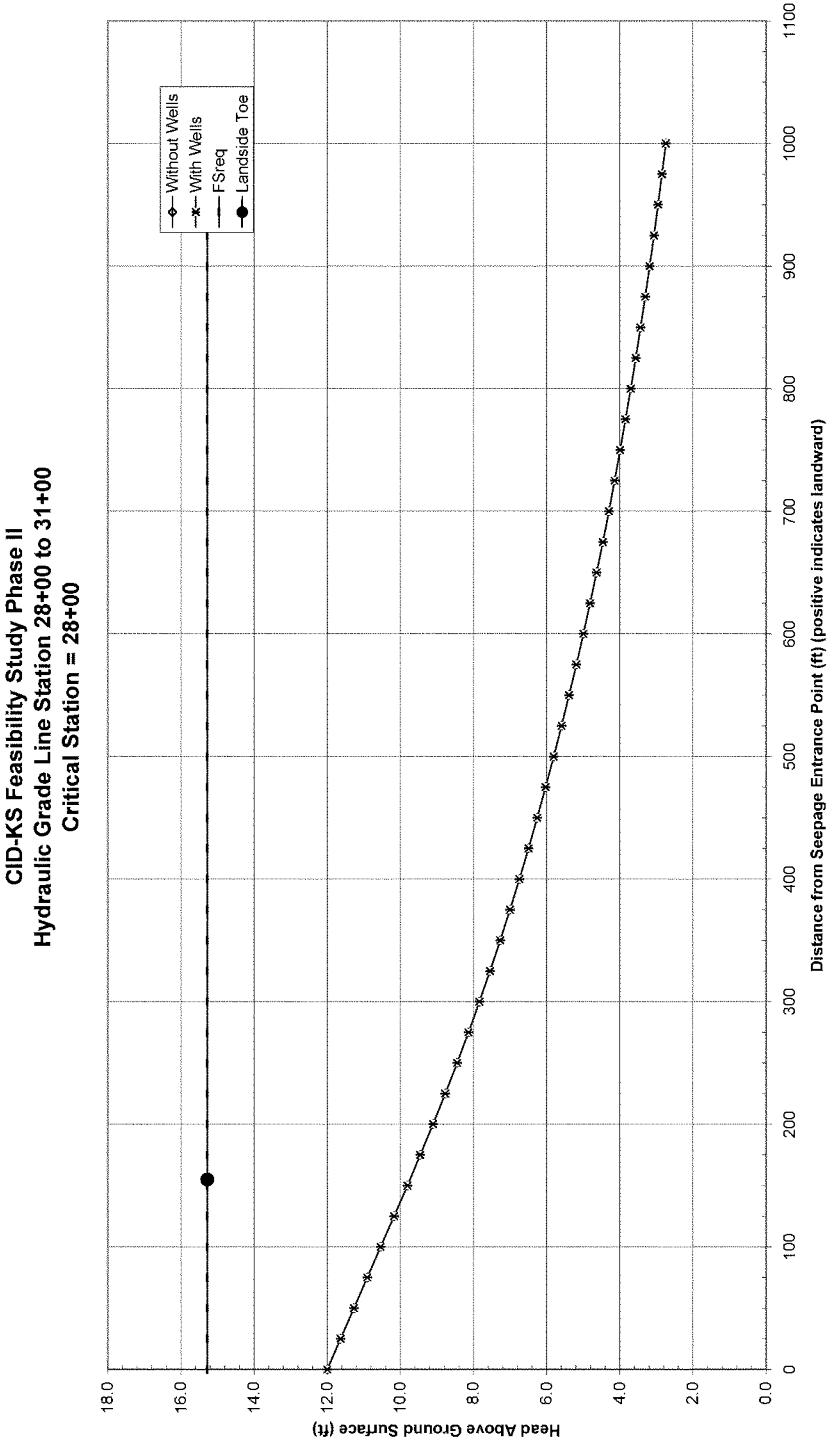
image well locations		
well	x'	y'
1	-65	3350
2	-65	3400
3	-65	3450
4	-65	3500
5	-65	3525
6	-65	3550
7	-65	3600
8	-65	3625
9	-75	3637
10	-75	3650
11	-85	3675
12	-100	3700
13	0	0
14	0	0
15	0	0
16	0	0
17	0	0
18	0	0
19	0	0
20	0	0

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

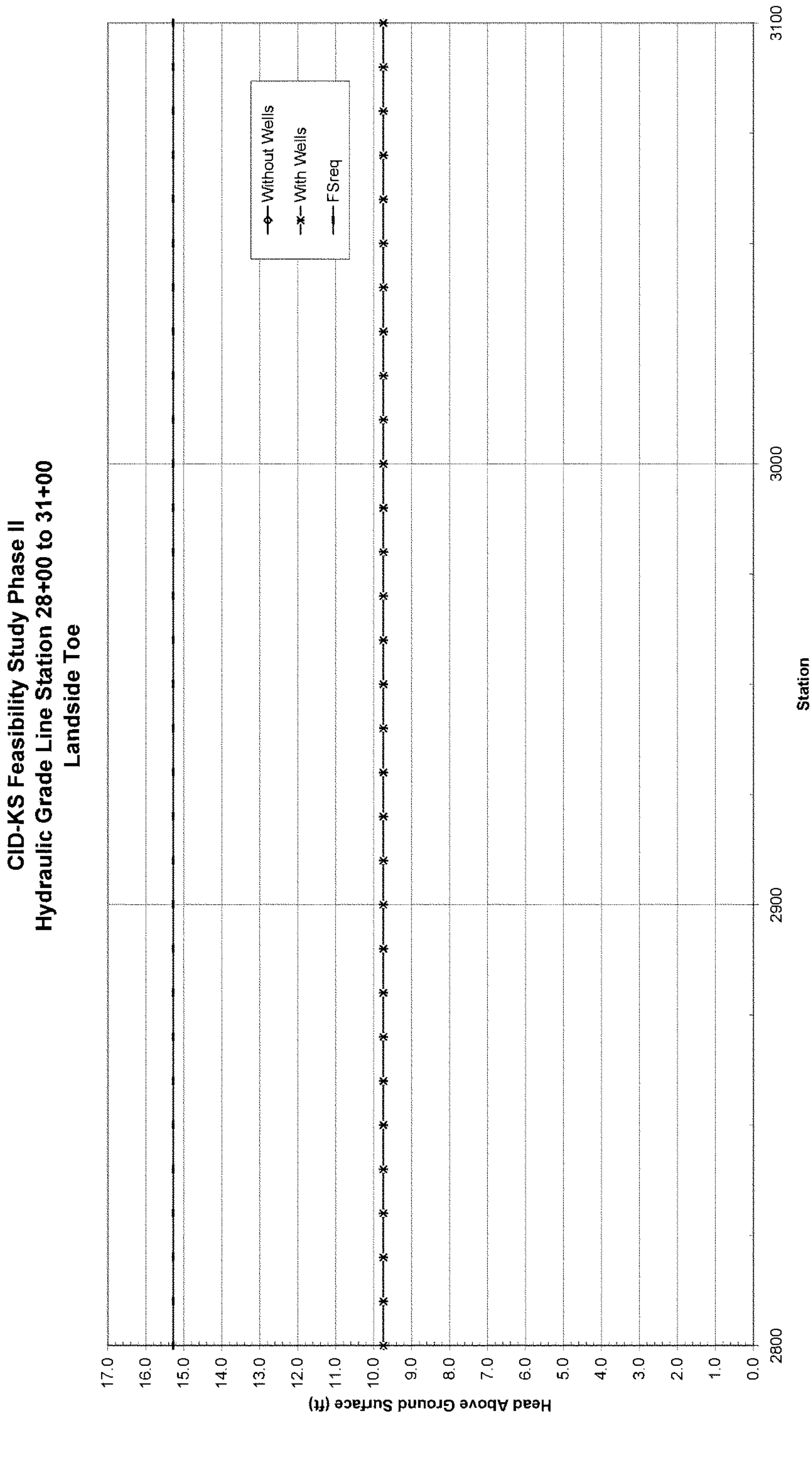
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	2800	12.0	0.0	12.0	15.28	1.00	0.41	2.04
2	25	2800	11.6	0.1	11.6	15.28	1.00	0.40	2.10
3	50	2800	11.3	0.1	11.3	15.28	1.00	0.39	2.17
4	75	2800	10.9	0.2	10.9	15.28	1.00	0.38	2.24
5	100	2800	10.5	0.2	10.5	15.28	1.00	0.36	2.32
6	125	2800	10.2	0.3	10.2	15.28	1.00	0.35	2.40
7	150	2800	9.8	0.3	9.8	15.28	1.00	0.34	2.49
8	175	2800	9.5	0.3	9.5	15.28	1.00	0.33	2.59
9	200	2800	9.1	0.4	9.1	15.28	1.00	0.31	2.68
10	225	2800	8.8	0.4	8.8	15.28	1.00	0.30	2.79
11	250	2800	8.4	0.5	8.4	15.28	1.00	0.29	2.89
12	275	2800	8.1	0.5	8.1	15.28	1.00	0.28	3.00
13	300	2800	7.8	0.5	7.8	15.28	1.00	0.27	3.12
14	325	2800	7.5	0.6	7.5	15.28	1.00	0.26	3.24
15	350	2800	7.3	0.6	7.3	15.28	1.00	0.25	3.36
16	375	2800	7.0	0.6	7.0	15.28	1.00	0.24	3.49
17	400	2800	6.7	0.6	6.7	15.28	1.00	0.23	3.62
18	425	2800	6.5	0.7	6.5	15.28	1.00	0.22	3.76
19	450	2800	6.3	0.7	6.3	15.28	1.00	0.22	3.91
20	475	2800	6.0	0.7	6.0	15.28	1.00	0.21	4.06
21	500	2800	5.8	0.7	5.8	15.28	1.00	0.20	4.21
22	525	2800	5.6	0.7	5.6	15.28	1.00	0.19	4.37
23	550	2800	5.4	0.7	5.4	15.28	1.00	0.19	4.54
24	575	2800	5.2	0.7	5.2	15.28	1.00	0.18	4.71
25	600	2800	5.0	0.7	5.0	15.28	1.00	0.17	4.89
26	625	2800	4.8	0.7	4.8	15.28	1.00	0.17	5.08
27	650	2800	4.6	0.8	4.6	15.28	1.00	0.16	5.27
28	675	2800	4.5	0.8	4.5	15.28	1.00	0.15	5.48
29	700	2800	4.3	0.8	4.3	15.28	1.00	0.15	5.69
30	725	2800	4.1	0.8	4.1	15.28	1.00	0.14	5.90
31	750	2800	4.0	0.8	4.0	15.28	1.00	0.14	6.13
32	775	2800	3.8	0.8	3.8	15.28	1.00	0.13	6.36
33	800	2800	3.7	0.8	3.7	15.28	1.00	0.13	6.61
34	825	2800	3.6	0.8	3.6	15.28	1.00	0.12	6.86
35	850	2800	3.4	0.8	3.4	15.28	1.00	0.12	7.12
36	875	2800	3.3	0.7	3.3	15.28	1.00	0.11	7.39
37	900	2800	3.2	0.7	3.2	15.28	1.00	0.11	7.68
38	925	2800	3.1	0.7	3.1	15.28	1.00	0.11	7.97
39	950	2800	3.0	0.7	3.0	15.28	1.00	0.10	8.27
40	975	2800	2.8	0.7	2.8	15.28	1.00	0.10	8.59
41	1000	2800	2.7	0.7	2.7	15.28	1.00	0.09	8.92

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	155	2700	9.7	0.2	9.7	15.28	1.00	0.34	2.51
2	155	2710	9.7	0.2	9.7	15.28	1.00	0.34	2.51
3	155	2720	9.7	0.3	9.7	15.28	1.00	0.34	2.51
4	155	2730	9.7	0.3	9.7	15.28	1.00	0.34	2.51
5	155	2740	9.7	0.3	9.7	15.28	1.00	0.34	2.51
6	155	2750	9.7	0.3	9.7	15.28	1.00	0.34	2.51
7	155	2760	9.7	0.3	9.7	15.28	1.00	0.34	2.51
8	155	2770	9.7	0.3	9.7	15.28	1.00	0.34	2.51
9	155	2780	9.7	0.3	9.7	15.28	1.00	0.34	2.51
10	155	2790	9.7	0.3	9.7	15.28	1.00	0.34	2.51
11	155	2800	9.7	0.3	9.7	15.28	1.00	0.34	2.51
12	155	2810	9.7	0.3	9.7	15.28	1.00	0.34	2.51
13	155	2820	9.7	0.3	9.7	15.28	1.00	0.34	2.51
14	155	2830	9.7	0.3	9.7	15.28	1.00	0.34	2.51
15	155	2840	9.7	0.4	9.7	15.28	1.00	0.34	2.51
16	155	2850	9.7	0.4	9.7	15.28	1.00	0.34	2.51
17	155	2860	9.7	0.4	9.7	15.28	1.00	0.34	2.51
18	155	2870	9.7	0.4	9.7	15.28	1.00	0.34	2.51
19	155	2880	9.7	0.4	9.7	15.28	1.00	0.34	2.51
20	155	2890	9.7	0.4	9.7	15.28	1.00	0.34	2.51
21	155	2900	9.7	0.4	9.7	15.28	1.00	0.34	2.51
22	155	2910	9.7	0.4	9.7	15.28	1.00	0.34	2.51
23	155	2920	9.7	0.4	9.7	15.28	1.00	0.34	2.51
24	155	2930	9.7	0.5	9.7	15.28	1.00	0.34	2.51
25	155	2940	9.7	0.5	9.7	15.28	1.00	0.34	2.51
26	155	2950	9.7	0.5	9.7	15.28	1.00	0.34	2.51
27	155	2960	9.7	0.5	9.7	15.28	1.00	0.34	2.51
28	155	2970	9.7	0.5	9.7	15.28	1.00	0.34	2.51
29	155	2980	9.7	0.5	9.7	15.28	1.00	0.34	2.51
30	155	2990	9.7	0.6	9.7	15.28	1.00	0.34	2.51
31	155	3000	9.7	0.6	9.7	15.28	1.00	0.34	2.51
32	155	3010	9.7	0.6	9.7	15.28	1.00	0.34	2.51
33	155	3020	9.7	0.6	9.7	15.28	1.00	0.34	2.51
34	155	3030	9.7	0.7	9.7	15.28	1.00	0.34	2.51
35	155	3040	9.7	0.7	9.7	15.28	1.00	0.34	2.51
36	155	3050	9.7	0.7	9.7	15.28	1.00	0.34	2.51
37	155	3060	9.7	0.7	9.7	15.28	1.00	0.34	2.51
38	155	3070	9.7	0.8	9.7	15.28	1.00	0.34	2.51
39	155	3080	9.7	0.8	9.7	15.28	1.00	0.34	2.51
40	155	3090	9.7	0.8	9.7	15.28	1.00	0.34	2.51
41	155	3100	9.7	0.9	9.7	15.28	1.00	0.34	2.51



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 28+00 to 31+00
Critical Station = 28+00
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
0
100
200
300
400
500
600
700
800
900
1000
1100


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 28+00 to 31+00
Landside Toe
Without Wells
With Wells
FSreq
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
2800
2900
3000
3100
Station

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	51	ft
h_p =	11.29	ft
TOL =	762	ft elevation
Landside =	748.0	ft elevation
Bottom Blanket =	721	ft elevation
blanket =	27.0	ft
z, Landside Toe =	155	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	11.20	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
1	65	3350	747.0	0.90	0.29	1.00
2	65	3400	747.0	0.82	0.26	1.00
3	65	3450	746.0	0.83	0.26	1.00
4	65	3500	746.0	0.75	0.24	1.00
5	65	3525	745.0	0.77	0.25	1.00
6	65	3550	745.0	0.77	0.25	1.00
7	65	3600	745.0	0.73	0.23	1.00
8	65	3625	745.0	0.69	0.22	1.00
9	75	3637	745.0	0.65	0.21	1.00
10	75	3650	745.0	0.66	0.21	1.00
11	85	3675	746.0	0.66	0.21	1.00
12	100	3700	746.0	0.71	0.23	1.00
13				0.00	0.00	0.00
14				0.00	0.00	0.00
15				0.00	0.00	0.00
16				0.00	0.00	0.00
17				0.00	0.00	0.00
18				0.00	0.00	0.00
19				0.00	0.00	0.00
20				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

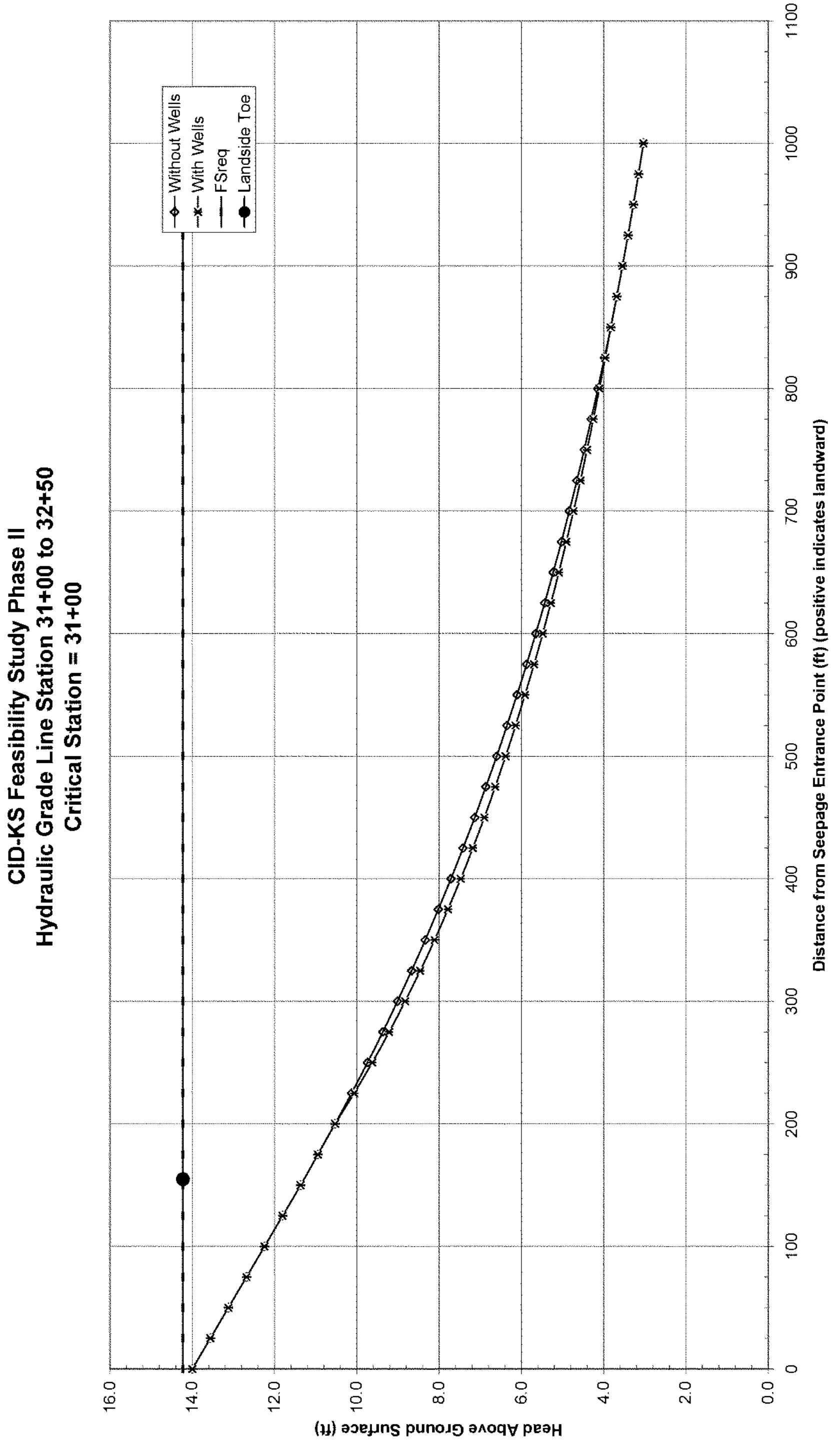
image well locations		
well	x'	y'
1	-65	3350
2	-65	3400
3	-65	3450
4	-65	3500
5	-65	3525
6	-65	3550
7	-65	3600
8	-65	3625
9	-75	3637
10	-75	3650
11	-85	3675
12	-100	3700
13	0	0
14	0	0
15	0	0
16	0	0
17	0	0
18	0	0
19	0	0
20	0	0

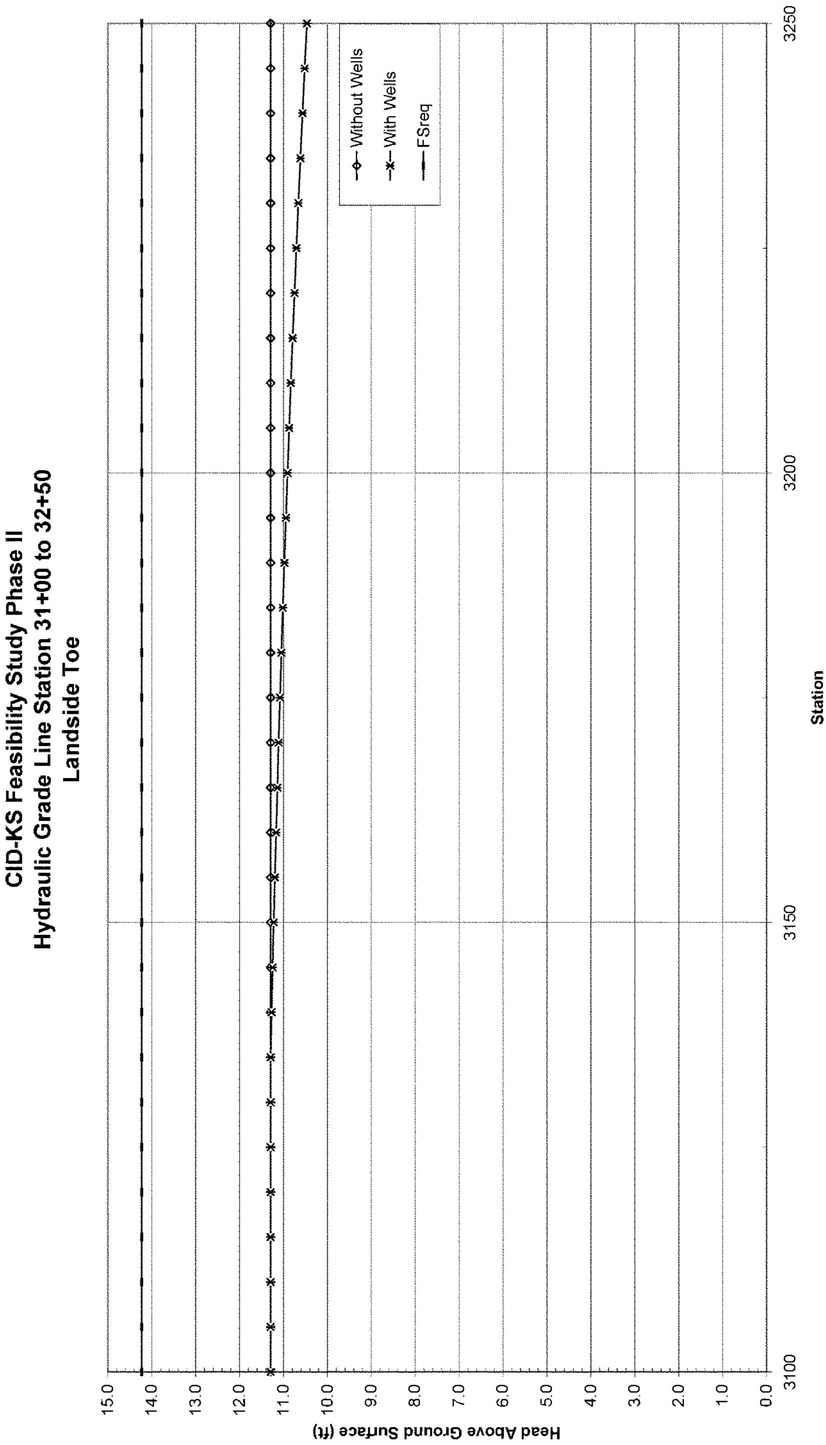
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	3100	14.0	0.0	14.0	14.22	1.00	0.52	1.63
2	25	3100	13.6	0.2	13.6	14.22	1.00	0.50	1.68
3	50	3100	13.1	0.3	13.1	14.22	1.00	0.49	1.73
4	75	3100	12.7	0.5	12.7	14.22	1.00	0.47	1.79
5	100	3100	12.2	0.6	12.2	14.22	1.00	0.45	1.86
6	125	3100	11.8	0.7	11.8	14.22	1.00	0.44	1.93
7	150	3100	11.4	0.8	11.4	14.22	1.00	0.42	2.00
8	175	3100	10.9	0.9	10.9	14.22	1.00	0.41	2.08
9	200	3100	10.5	1.0	10.5	14.22	1.00	0.39	2.16
10	225	3100	10.1	1.1	10.1	14.22	1.00	0.37	2.26
11	250	3100	9.7	1.1	9.6	14.22	1.00	0.36	2.36
12	275	3100	9.4	1.2	9.2	14.22	1.00	0.34	2.47
13	300	3100	9.0	1.2	8.8	14.22	1.00	0.33	2.58
14	325	3100	8.7	1.2	8.5	14.22	1.00	0.31	2.69
15	350	3100	8.3	1.2	8.1	14.22	1.00	0.30	2.81
16	375	3100	8.0	1.2	7.8	14.22	1.00	0.29	2.92
17	400	3100	7.7	1.2	7.5	14.22	1.00	0.28	3.05
18	425	3100	7.4	1.2	7.2	14.22	1.00	0.27	3.17
19	450	3100	7.1	1.2	6.9	14.22	1.00	0.26	3.30
20	475	3100	6.9	1.2	6.6	14.22	1.00	0.25	3.43
21	500	3100	6.6	1.2	6.4	14.22	1.00	0.24	3.57
22	525	3100	6.3	1.2	6.1	14.22	1.00	0.23	3.71
23	550	3100	6.1	1.2	5.9	14.22	1.00	0.22	3.85
24	575	3100	5.9	1.2	5.7	14.22	1.00	0.21	4.00
25	600	3100	5.7	1.2	5.5	14.22	1.00	0.20	4.15
26	625	3100	5.4	1.2	5.3	14.22	1.00	0.20	4.31
27	650	3100	5.2	1.1	5.1	14.22	1.00	0.19	4.47
28	675	3100	5.0	1.1	4.9	14.22	1.00	0.18	4.64
29	700	3100	4.8	1.1	4.7	14.22	1.00	0.18	4.81
30	725	3100	4.7	1.1	4.6	14.22	1.00	0.17	4.99
31	750	3100	4.5	1.1	4.4	14.22	1.00	0.16	5.17
32	775	3100	4.3	1.1	4.3	14.22	1.00	0.16	5.35
33	800	3100	4.1	1.0	4.1	14.22	1.00	0.15	5.55
34	825	3100	4.0	1.0	4.0	14.22	1.00	0.15	5.74
35	850	3100	3.8	1.0	3.8	14.22	1.00	0.14	5.95
36	875	3100	3.7	1.0	3.7	14.22	1.00	0.14	6.18
37	900	3100	3.5	1.0	3.5	14.22	1.00	0.13	6.42
38	925	3100	3.4	1.0	3.4	14.22	1.00	0.13	6.68
39	950	3100	3.3	0.9	3.3	14.22	1.00	0.12	6.94
40	975	3100	3.2	0.9	3.2	14.22	1.00	0.12	7.22
41	1000	3100	3.0	0.9	3.0	14.22	1.00	0.11	7.51

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	155	3090	11.3	0.8	11.3	14.22	1.00	0.42	2.02
2	155	3095	11.3	0.8	11.3	14.22	1.00	0.42	2.02
3	155	3100	11.3	0.8	11.3	14.22	1.00	0.42	2.02
4	155	3105	11.3	0.9	11.3	14.22	1.00	0.42	2.02
5	155	3110	11.3	0.9	11.3	14.22	1.00	0.42	2.02
6	155	3115	11.3	0.9	11.3	14.22	1.00	0.42	2.02
7	155	3120	11.3	0.9	11.3	14.22	1.00	0.42	2.02
8	155	3125	11.3	0.9	11.3	14.22	1.00	0.42	2.02
9	155	3130	11.3	1.0	11.3	14.22	1.00	0.42	2.02
10	155	3135	11.3	1.0	11.3	14.22	1.00	0.42	2.02
11	155	3140	11.3	1.0	11.3	14.22	1.00	0.42	2.02
12	155	3145	11.3	1.0	11.2	14.22	1.00	0.42	2.02
13	155	3150	11.3	1.1	11.2	14.22	1.00	0.42	2.03
14	155	3155	11.3	1.1	11.2	14.22	1.00	0.41	2.03
15	155	3160	11.3	1.1	11.2	14.22	1.00	0.41	2.04
16	155	3165	11.3	1.2	11.1	14.22	1.00	0.41	2.04
17	155	3170	11.3	1.2	11.1	14.22	1.00	0.41	2.05
18	155	3175	11.3	1.2	11.1	14.22	1.00	0.41	2.05
19	155	3180	11.3	1.2	11.0	14.22	1.00	0.41	2.06
20	155	3185	11.3	1.3	11.0	14.22	1.00	0.41	2.07
21	155	3190	11.3	1.3	11.0	14.22	1.00	0.41	2.07
22	155	3195	11.3	1.3	10.9	14.22	1.00	0.41	2.08
23	155	3200	11.3	1.4	10.9	14.22	1.00	0.40	2.09
24	155	3205	11.3	1.4	10.9	14.22	1.00	0.40	2.09
25	155	3210	11.3	1.5	10.8	14.22	1.00	0.40	2.10
26	155	3215	11.3	1.5	10.8	14.22	1.00	0.40	2.11
27	155	3220	11.3	1.5	10.7	14.22	1.00	0.40	2.12
28	155	3225	11.3	1.6	10.7	14.22	1.00	0.40	2.13
29	155	3230	11.3	1.6	10.7	14.22	1.00	0.39	2.13
30	155	3235	11.3	1.7	10.6	14.22	1.00	0.39	2.14
31	155	3240	11.3	1.7	10.6	14.22	1.00	0.39	2.15
32	155	3245	11.3	1.8	10.5	14.22	1.00	0.39	2.16
33	155	3250	11.3	1.8	10.5	14.22	1.00	0.38	2.17
34	155	3255	11.3	1.9	10.4	14.22	1.00	0.38	2.19
35	155	3260	11.3	1.9	10.4	14.22	1.00	0.38	2.20
36	155	3265	11.3	2.0	10.3	14.22	1.00	0.38	2.21
37	155	3270	11.3	2.0	10.2	14.22	1.00	0.38	2.22
38	155	3275	11.3	2.1	10.2	14.22	1.00	0.38	2.24
39	155	3280	11.3	2.2	10.1	14.22	1.00	0.37	2.25
40	155	3285	11.3	2.2	10.1	14.22	1.00	0.37	2.26
41	155	3290	11.3	2.3	10.0	14.22	1.00	0.37	2.28


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 31+00 to 32+50
Critical Station = 31+00
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 31+00 to 32+50
Landside Toe
Head Above Ground Surface (ft)
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
3100
3150
3200
3250
Station
Without Wells
With Wells
FSreq

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	51	ft
h_0 =	13.59	ft
TOL =	762	ft elevation
Landside =	748.0	ft elevation
Bottom Blanket =	721	ft elevation
blanket =	25.0	ft
z, Landside Toe =	110	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	11.44	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
1	65	3350	747.0	0.93	0.29	1.00
2	65	3400	747.0	0.84	0.27	1.00
3	65	3450	746.0	0.85	0.27	1.00
4	65	3500	746.0	0.77	0.25	1.00
5	65	3525	745.0	0.79	0.25	1.00
6	65	3550	745.0	0.79	0.25	1.00
7	65	3600	745.0	0.75	0.24	1.00
8	65	3625	745.0	0.70	0.22	1.00
9	75	3637	745.0	0.66	0.21	1.00
10	75	3650	745.0	0.68	0.22	1.00
11	85	3675	746.0	0.67	0.21	1.00
12	100	3700	746.0	0.72	0.23	1.00
13				0.00	0.00	0.00
14				0.00	0.00	0.00
15				0.00	0.00	0.00
16				0.00	0.00	0.00
17				0.00	0.00	0.00
18				0.00	0.00	0.00
19				0.00	0.00	0.00
20				0.00	0.00	0.00

image well locations		
well	x'	y'
1	-65	3350
2	-65	3400
3	-65	3450
4	-65	3500
5	-65	3525
6	-65	3550
7	-65	3600
8	-65	3625
9	-75	3637
10	-75	3650
11	-85	3675
12	-100	3700
13	0	0
14	0	0
15	0	0
16	0	0
17	0	0
18	0	0
19	0	0
20	0	0

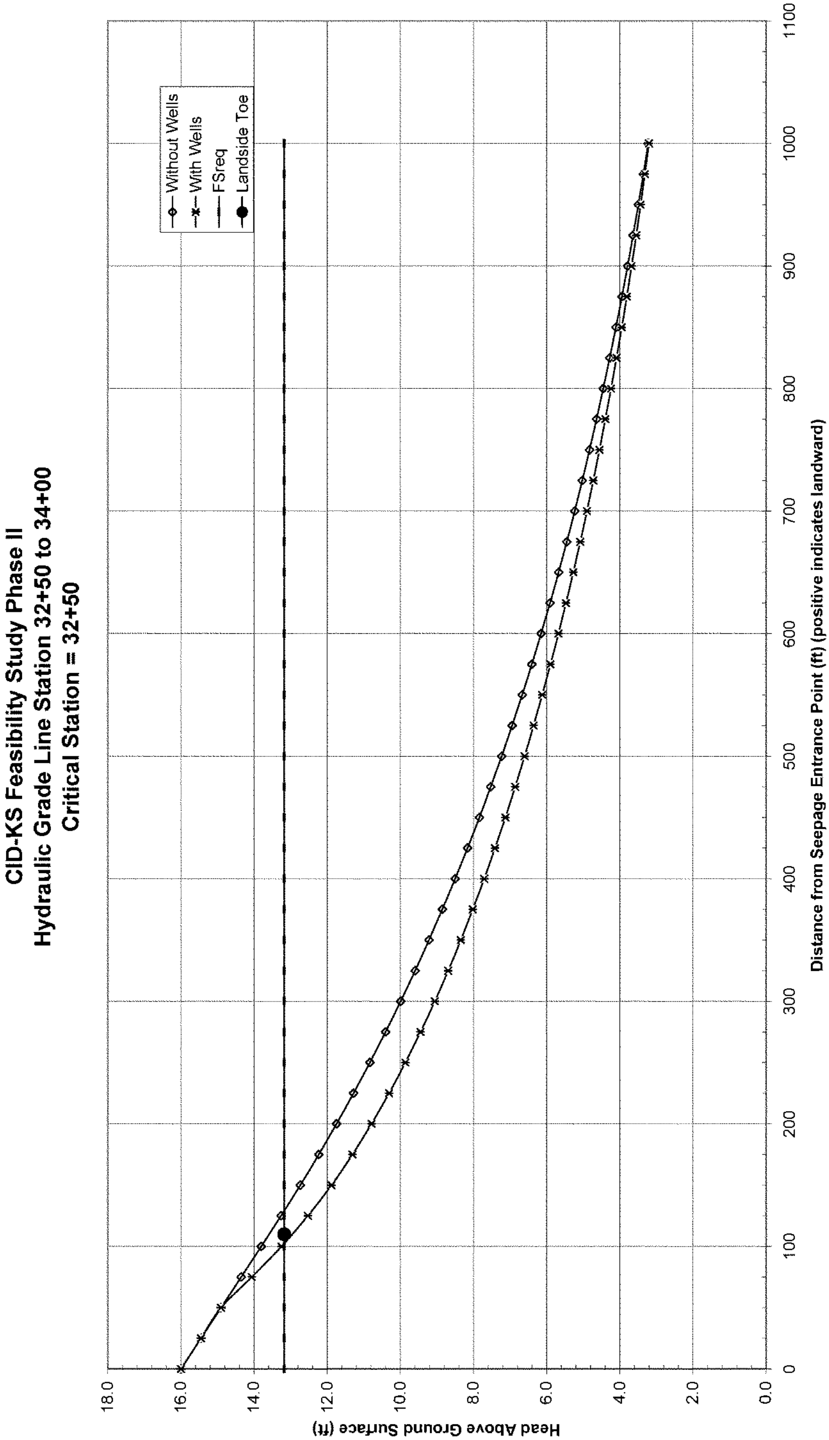
1.00 **<---Input H_w AVG after any changes are made to well parameters**

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

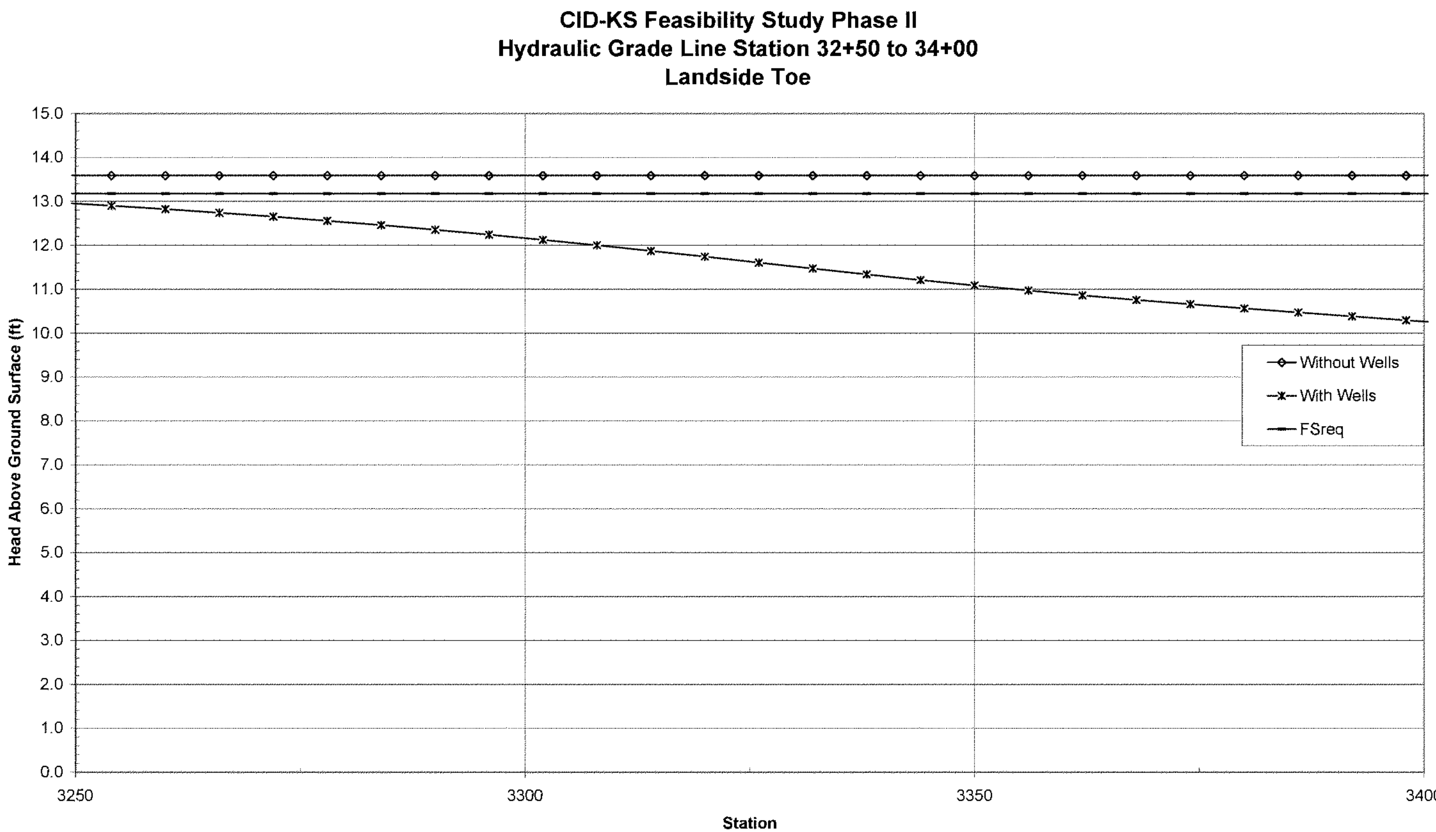
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	3250	16.0	0.0	16.0	13.17	1.00	0.64	1.32
2	25	3250	15.5	0.5	15.5	13.17	1.00	0.62	1.36
3	50	3250	14.9	0.9	14.9	13.17	1.00	0.60	1.41
4	75	3250	14.4	1.3	14.1	13.17	1.00	0.56	1.50
5	100	3250	13.8	1.6	13.2	13.17	1.00	0.53	1.59
6	125	3250	13.3	1.7	12.5	13.17	1.00	0.50	1.68
7	150	3250	12.7	1.9	11.9	13.17	1.00	0.46	1.77
8	175	3250	12.2	1.9	11.3	13.17	1.00	0.45	1.86
9	200	3250	11.7	2.0	10.8	13.17	1.00	0.43	1.95
10	225	3250	11.3	2.0	10.3	13.17	1.00	0.41	2.04
11	250	3250	10.8	2.0	9.9	13.17	1.00	0.39	2.14
12	275	3250	10.4	2.0	9.4	13.17	1.00	0.38	2.23
13	300	3250	10.0	1.9	9.1	13.17	1.00	0.36	2.33
14	325	3250	9.6	1.9	8.7	13.17	1.00	0.35	2.42
15	350	3250	9.2	1.9	8.3	13.17	1.00	0.33	2.52
16	375	3250	8.9	1.8	8.0	13.17	1.00	0.32	2.63
17	400	3250	8.5	1.8	7.7	13.17	1.00	0.31	2.73
18	425	3250	8.2	1.8	7.4	13.17	1.00	0.30	2.84
19	450	3250	7.8	1.7	7.1	13.17	1.00	0.29	2.96
20	475	3250	7.5	1.7	6.9	13.17	1.00	0.27	3.07
21	500	3250	7.2	1.6	6.6	13.17	1.00	0.26	3.19
22	525	3250	6.9	1.6	6.4	13.17	1.00	0.25	3.32
23	550	3250	6.7	1.6	6.1	13.17	1.00	0.24	3.44
24	575	3250	6.4	1.5	5.9	13.17	1.00	0.24	3.58
25	600	3250	6.2	1.5	5.7	13.17	1.00	0.23	3.71
26	625	3250	5.9	1.4	5.5	13.17	1.00	0.22	3.85
27	650	3250	5.7	1.4	5.3	13.17	1.00	0.21	4.00
28	675	3250	5.4	1.4	5.1	13.17	1.00	0.20	4.15
29	700	3250	5.2	1.3	4.9	13.17	1.00	0.20	4.30
30	725	3250	5.0	1.3	4.7	13.17	1.00	0.19	4.46
31	750	3250	4.8	1.3	4.6	13.17	1.00	0.18	4.63
32	775	3250	4.6	1.2	4.4	13.17	1.00	0.18	4.80
33	800	3250	4.5	1.2	4.2	13.17	1.00	0.17	4.97
34	825	3250	4.3	1.2	4.1	13.17	1.00	0.16	5.15
35	850	3250	4.1	1.2	3.9	13.17	1.00	0.16	5.34
36	875	3250	3.9	1.1	3.8	13.17	1.00	0.15	5.53
37	900	3250	3.8	1.1	3.7	13.17	1.00	0.15	5.73
38	925	3250	3.6	1.1	3.6	13.17	1.00	0.14	5.93
39	950	3250	3.5	1.1	3.4	13.17	1.00	0.14	6.14
40	975	3250	3.4	1.0	3.3	13.17	1.00	0.13	6.36
41	1000	3250	3.2	1.0	3.2	13.17	1.00	0.13	6.58

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	110	3200	13.6	1.2	13.4	13.17	1.00	0.54	1.57
2	110	3206	13.6	1.2	13.4	13.17	1.00	0.54	1.57
3	110	3212	13.6	1.3	13.3	13.17	1.00	0.53	1.58
4	110	3218	13.6	1.3	13.3	13.17	1.00	0.53	1.59
5	110	3224	13.6	1.4	13.2	13.17	1.00	0.53	1.59
6	110	3230	13.6	1.4	13.2	13.17	1.00	0.53	1.60
7	110	3236	13.6	1.5	13.1	13.17	1.00	0.52	1.61
8	110	3242	13.6	1.5	13.0	13.17	1.00	0.52	1.62
9	110	3248	13.6	1.6	13.0	13.17	1.00	0.52	1.62
10	110	3254	13.6	1.7	12.9	13.17	1.00	0.52	1.63
11	110	3260	13.6	1.8	12.8	13.17	1.00	0.51	1.64
12	110	3266	13.6	1.8	12.7	13.17	1.00	0.51	1.65
13	110	3272	13.6	1.9	12.7	13.17	1.00	0.51	1.67
14	110	3278	13.6	2.0	12.6	13.17	1.00	0.50	1.68
15	110	3284	13.6	2.1	12.5	13.17	1.00	0.50	1.69
16	110	3290	13.6	2.2	12.4	13.17	1.00	0.49	1.71
17	110	3296	13.6	2.3	12.2	13.17	1.00	0.49	1.72
18	110	3302	13.6	2.5	12.1	13.17	1.00	0.48	1.74
19	110	3308	13.6	2.6	12.0	13.17	1.00	0.48	1.76
20	110	3314	13.6	2.7	11.9	13.17	1.00	0.47	1.78
21	110	3320	13.6	2.8	11.7	13.17	1.00	0.47	1.80
22	110	3326	13.6	3.0	11.6	13.17	1.00	0.46	1.82
23	110	3332	13.6	3.1	11.5	13.17	1.00	0.46	1.84
24	110	3338	13.6	3.3	11.3	13.17	1.00	0.45	1.86
25	110	3344	13.6	3.4	11.2	13.17	1.00	0.45	1.88
26	110	3350	13.6	3.5	11.1	13.17	1.00	0.44	1.90
27	110	3356	13.6	3.6	11.0	13.17	1.00	0.44	1.92
28	110	3362	13.6	3.7	10.9	13.17	1.00	0.43	1.94
29	110	3368	13.6	3.8	10.8	13.17	1.00	0.43	1.96
30	110	3374	13.6	3.9	10.7	13.17	1.00	0.43	1.98
31	110	3380	13.6	4.0	10.6	13.17	1.00	0.42	2.00
32	110	3386	13.6	4.1	10.5	13.17	1.00	0.42	2.01
33	110	3392	13.6	4.2	10.4	13.17	1.00	0.42	2.03
34	110	3398	13.6	4.3	10.3	13.17	1.00	0.41	2.05
35	110	3404	13.6	4.4	10.2	13.17	1.00	0.41	2.06
36	110	3410	13.6	4.5	10.1	13.17	1.00	0.41	2.08
37	110	3416	13.6	4.5	10.1	13.17	1.00	0.40	2.10
38	110	3422	13.6	4.6	10.0	13.17	1.00	0.40	2.11
39	110	3428	13.6	4.7	9.9	13.17	1.00	0.40	2.13
40	110	3434	13.6	4.8	9.8	13.17	1.00	0.39	2.14
41	110	3440	13.6	4.8	9.8	13.17	1.00	0.39	2.16



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 32+50 to 34+00
Critical Station = 32+50
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 32+50 to 34+00
Landside Toe
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Head Above Ground Surface (ft)
3250
3300
3350
3400
Station
Without Wells
With Wells
FSreq

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	51	ft
h_o =	15.95	ft
TOL =	762	ft elevation
Landside =	745.0	ft elevation
Bottom Blanket =	721	ft elevation
blanket =	24.0	ft
z, Landside Toe =	65	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	12.58	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
1	65	3350	747.0	1.03	0.33	1.00
2	65	3400	747.0	0.93	0.30	1.00
3	65	3450	746.0	0.94	0.30	1.00
4	65	3500	746.0	0.85	0.27	1.00
5	65	3525	745.0	0.86	0.27	1.00
6	65	3550	745.0	0.86	0.27	1.00
7	65	3600	745.0	0.82	0.26	1.00
8	65	3625	745.0	0.77	0.24	1.00
9	75	3637	745.0	0.72	0.23	1.00
10	75	3650	745.0	0.74	0.24	1.00
11	85	3675	746.0	0.74	0.24	1.00
12	100	3700	746.0	0.80	0.25	1.00
13				0.00	0.00	0.00
14				0.00	0.00	0.00
15				0.00	0.00	0.00
16				0.00	0.00	0.00
17				0.00	0.00	0.00
18				0.00	0.00	0.00
19				0.00	0.00	0.00
20				0.00	0.00	0.00
						1.00

<--Input H_w AVG after any changes are made to well parameters

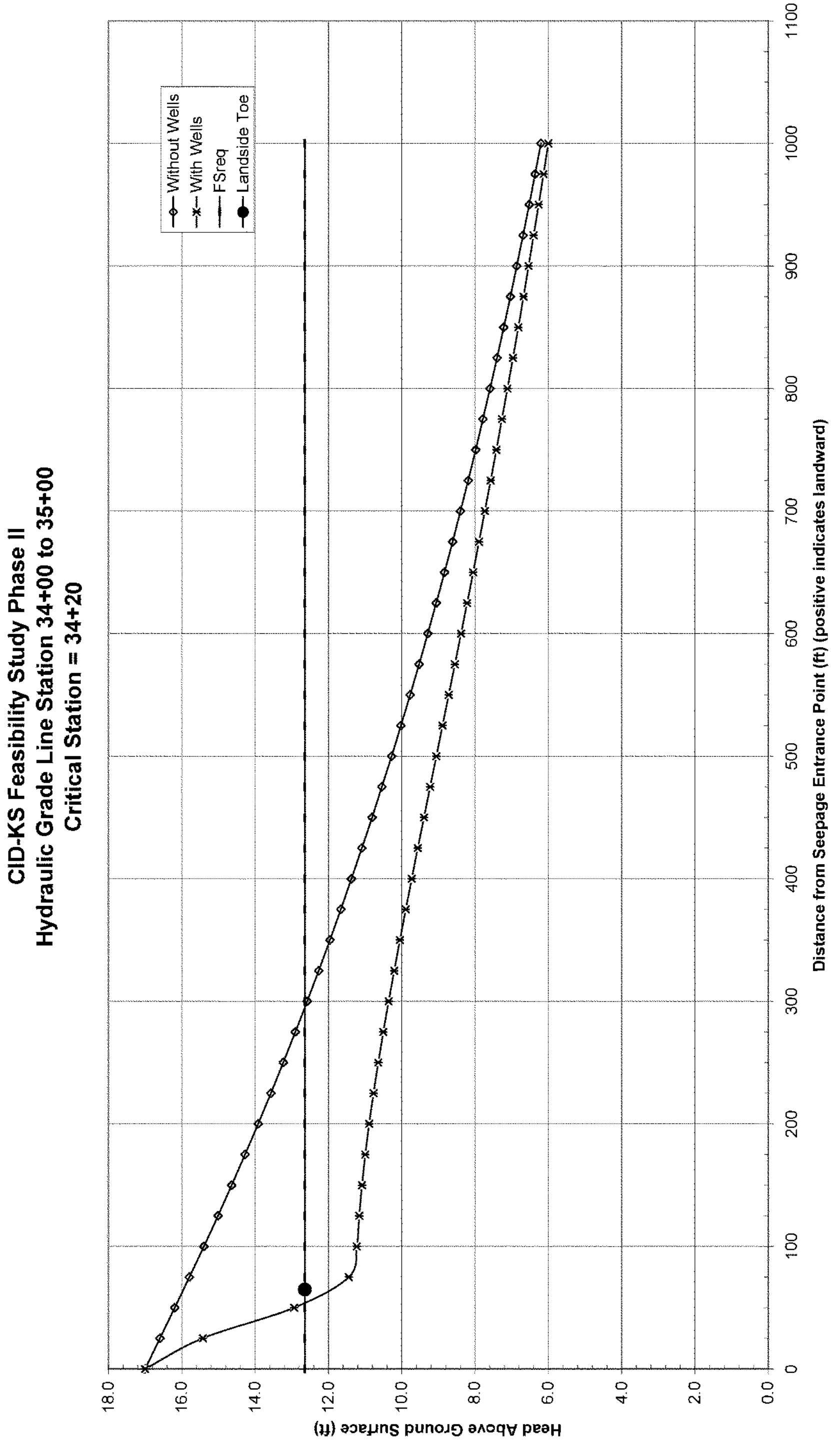
image well locations		
well	x'	y'
1	-65	3350
2	-65	3400
3	-65	3450
4	-65	3500
5	-65	3525
6	-65	3550
7	-65	3600
8	-65	3625
9	-75	3637
10	-75	3650
11	-85	3675
12	-100	3700
13	0	0
14	0	0
15	0	0
16	0	0
17	0	0
18	0	0
19	0	0
20	0	0

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

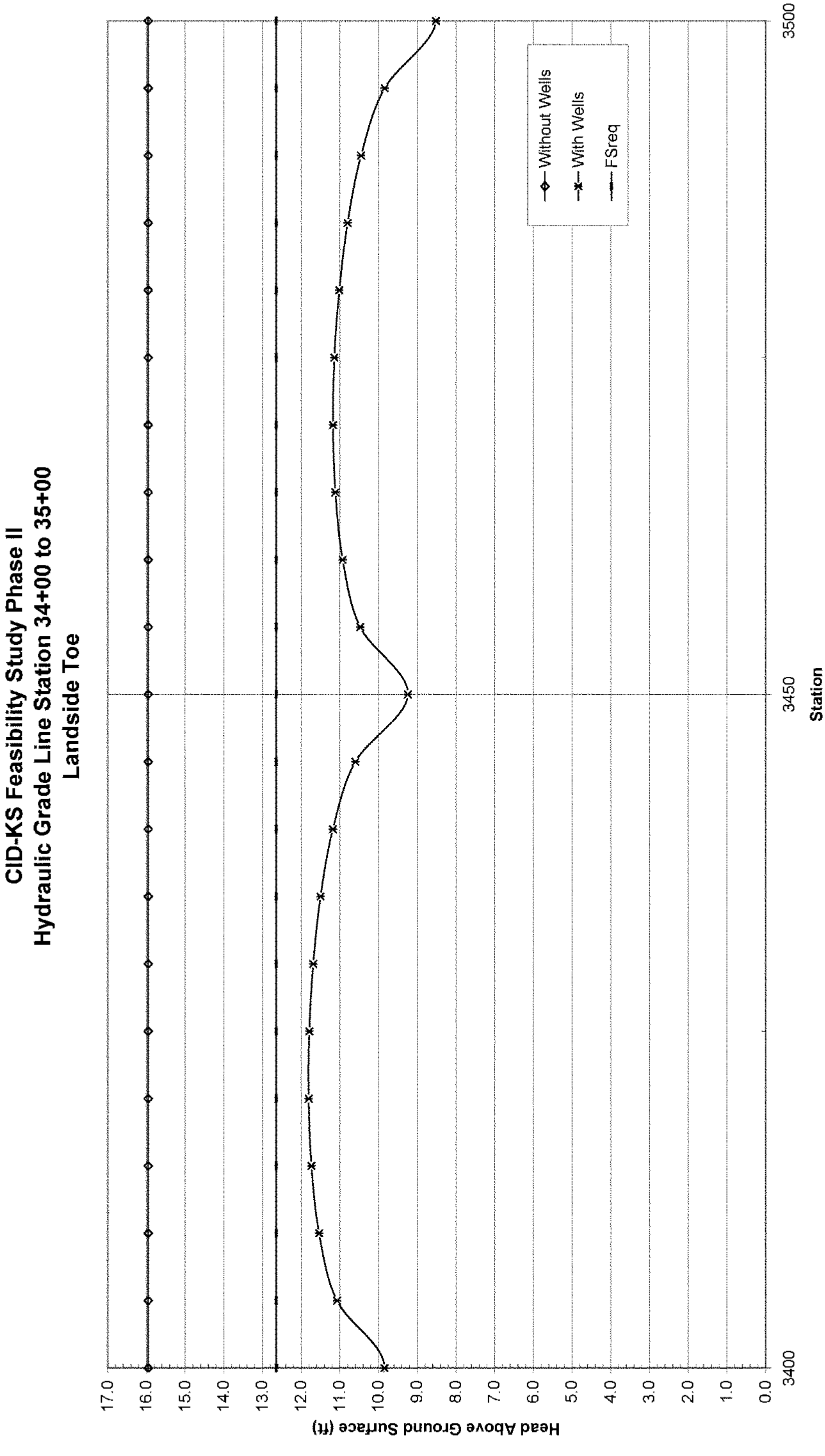
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	3420	17.0	0.0	17.0	12.64	1.00	0.71	1.19
2	25	3420	16.6	2.2	15.4	12.64	1.00	0.64	1.31
3	50	3420	16.2	4.3	12.9	12.64	1.00	0.54	1.56
4	75	3420	15.8	5.3	11.4	12.64	1.00	0.46	1.77
5	100	3420	15.4	5.2	11.2	12.64	1.00	0.47	1.80
6	125	3420	15.0	4.9	11.2	12.64	1.00	0.47	1.81
7	150	3420	14.6	4.6	11.1	12.64	1.00	0.46	1.83
8	175	3420	14.3	4.3	11.0	12.64	1.00	0.46	1.84
9	200	3420	13.9	4.0	10.9	12.64	1.00	0.45	1.86
10	225	3420	13.6	3.8	10.8	12.64	1.00	0.45	1.88
11	250	3420	13.2	3.6	10.6	12.64	1.00	0.44	1.90
12	275	3420	12.9	3.4	10.5	12.64	1.00	0.44	1.93
13	300	3420	12.6	3.2	10.4	12.64	1.00	0.43	1.95
14	325	3420	12.3	3.1	10.2	12.64	1.00	0.43	1.98
15	350	3420	12.0	2.9	10.0	12.64	1.00	0.42	2.01
16	375	3420	11.7	2.8	9.9	12.64	1.00	0.41	2.05
17	400	3420	11.4	2.6	9.7	12.64	1.00	0.41	2.08
18	425	3420	11.1	2.5	9.6	12.64	1.00	0.40	2.12
19	450	3420	10.8	2.4	9.4	12.64	1.00	0.39	2.15
20	475	3420	10.5	2.3	9.2	12.64	1.00	0.38	2.19
21	500	3420	10.3	2.2	9.1	12.64	1.00	0.38	2.23
22	525	3420	10.0	2.1	8.9	12.64	1.00	0.37	2.28
23	550	3420	9.8	2.1	8.7	12.64	1.00	0.36	2.32
24	575	3420	9.5	2.0	8.5	12.64	1.00	0.36	2.37
25	600	3420	9.3	1.9	8.4	12.64	1.00	0.35	2.41
26	625	3420	9.1	1.8	8.2	12.64	1.00	0.34	2.46
27	650	3420	8.8	1.8	8.1	12.64	1.00	0.34	2.51
28	675	3420	8.6	1.7	7.9	12.64	1.00	0.33	2.56
29	700	3420	8.4	1.7	7.7	12.64	1.00	0.32	2.62
30	725	3420	8.2	1.6	7.6	12.64	1.00	0.32	2.67
31	750	3420	8.0	1.6	7.4	12.64	1.00	0.31	2.73
32	775	3420	7.8	1.5	7.3	12.64	1.00	0.30	2.78
33	800	3420	7.6	1.5	7.1	12.64	1.00	0.30	2.84
34	825	3420	7.4	1.4	7.0	12.64	1.00	0.29	2.90
35	850	3420	7.2	1.4	6.8	12.64	1.00	0.28	2.97
36	875	3420	7.0	1.4	6.7	12.64	1.00	0.28	3.03
37	900	3420	6.9	1.3	6.5	12.64	1.00	0.27	3.09
38	925	3420	6.7	1.3	6.4	12.64	1.00	0.27	3.16
39	950	3420	6.5	1.3	6.3	12.64	1.00	0.26	3.23
40	975	3420	6.4	1.2	6.1	12.64	1.00	0.26	3.30
41	1000	3420	6.2	1.2	6.0	12.64	1.00	0.25	3.37

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	65	3390	16.0	5.2	11.8	12.64	1.00	0.49	1.72
2	65	3395	16.0	5.7	11.2	12.64	1.00	0.47	1.81
3	65	3400	16.0	7.1	9.8	12.64	1.00	0.41	2.05
4	65	3405	16.0	5.9	11.1	12.64	1.00	0.46	1.83
5	65	3410	16.0	5.4	11.5	12.64	1.00	0.48	1.75
6	65	3415	16.0	5.2	11.7	12.64	1.00	0.49	1.72
7	65	3420	16.0	5.1	11.8	12.64	1.00	0.49	1.71
8	65	3425	16.0	5.2	11.8	12.64	1.00	0.49	1.72
9	65	3430	16.0	5.3	11.7	12.64	1.00	0.49	1.73
10	65	3435	16.0	5.5	11.5	12.64	1.00	0.48	1.76
11	65	3440	16.0	5.8	11.2	12.64	1.00	0.47	1.81
12	65	3445	16.0	6.4	10.6	12.64	1.00	0.44	1.91
13	65	3450	16.0	7.7	9.2	12.64	1.00	0.39	2.19
14	65	3455	16.0	6.5	10.5	12.64	1.00	0.44	1.93
15	65	3460	16.0	6.0	10.9	12.64	1.00	0.46	1.85
16	65	3465	16.0	5.8	11.1	12.64	1.00	0.46	1.82
17	65	3470	16.0	5.8	11.2	12.64	1.00	0.47	1.81
18	65	3475	16.0	5.8	11.1	12.64	1.00	0.46	1.82
19	65	3480	16.0	5.9	11.0	12.64	1.00	0.46	1.84
20	65	3485	16.0	6.2	10.8	12.64	1.00	0.45	1.87
21	65	3490	16.0	6.5	10.5	12.64	1.00	0.44	1.94
22	65	3495	16.0	7.1	9.8	12.64	1.00	0.41	2.05
23	65	3500	16.0	8.4	8.5	12.64	1.00	0.35	2.38
24	65	3505	16.0	7.5	9.5	12.64	1.00	0.40	2.13
25	65	3510	16.0	7.2	9.7	12.64	1.00	0.41	2.08
26	65	3515	16.0	7.3	9.6	12.64	1.00	0.40	2.10
27	65	3520	16.0	7.8	9.2	12.64	1.00	0.38	2.20
28	65	3525	16.0	9.0	8.0	12.64	1.00	0.33	2.53
29	65	3530	16.0	7.8	9.1	12.64	1.00	0.38	2.22
30	65	3535	16.0	7.5	9.5	12.64	1.00	0.39	2.14
31	65	3540	16.0	7.5	9.5	12.64	1.00	0.39	2.14
32	65	3545	16.0	7.8	9.1	12.64	1.00	0.38	2.21
33	65	3550	16.0	8.9	8.1	12.64	1.00	0.34	2.51
34	65	3555	16.0	7.6	9.3	12.64	1.00	0.39	2.17
35	65	3560	16.0	7.1	9.8	12.64	1.00	0.41	2.06
36	65	3565	16.0	6.9	10.1	12.64	1.00	0.42	2.00
37	65	3570	16.0	6.7	10.2	12.64	1.00	0.43	1.96
38	65	3575	16.0	6.7	10.2	12.64	1.00	0.43	1.98
39	65	3580	16.0	6.8	10.2	12.64	1.00	0.42	1.99
40	65	3585	16.0	7.0	10.0	12.64	1.00	0.42	2.03
41	65	3590	16.0	7.3	9.7	12.64	1.00	0.40	2.09



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 34+00 to 35+00
Critical Station = 34+20
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100
Distance from Seepage Entrance Point (ft) (positive indicates landward)



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 34+00 to 35+00
Landside Toe
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
3400
3450
3500
Station
Without Wells
With Wells
FSreq

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	51	ft
h_o =	16.87	ft
TOL =	762	ft elevation
Landside =	744.0	ft elevation
Bottom Blanket =	721	ft elevation
blanket =	23.0	ft
z, Landside Toe =	65	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	12.51	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
1	65	3350	747.0	1.02	0.32	1.00
2	65	3400	747.0	0.93	0.30	1.00
3	65	3450	746.0	0.93	0.30	1.00
4	65	3500	746.0	0.84	0.27	1.00
5	65	3525	745.0	0.86	0.27	1.00
6	65	3550	745.0	0.86	0.27	1.00
7	65	3600	745.0	0.82	0.26	1.00
8	65	3625	745.0	0.76	0.24	1.00
9	75	3637	745.0	0.72	0.23	1.00
10	75	3650	745.0	0.74	0.23	1.00
11	85	3575	746.0	0.74	0.23	1.00
12	100	3700	746.0	0.79	0.25	1.00
13				0.00	0.00	0.00
14				0.00	0.00	0.00
15				0.00	0.00	0.00
16				0.00	0.00	0.00
17				0.00	0.00	0.00
18				0.00	0.00	0.00
19				0.00	0.00	0.00
20				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

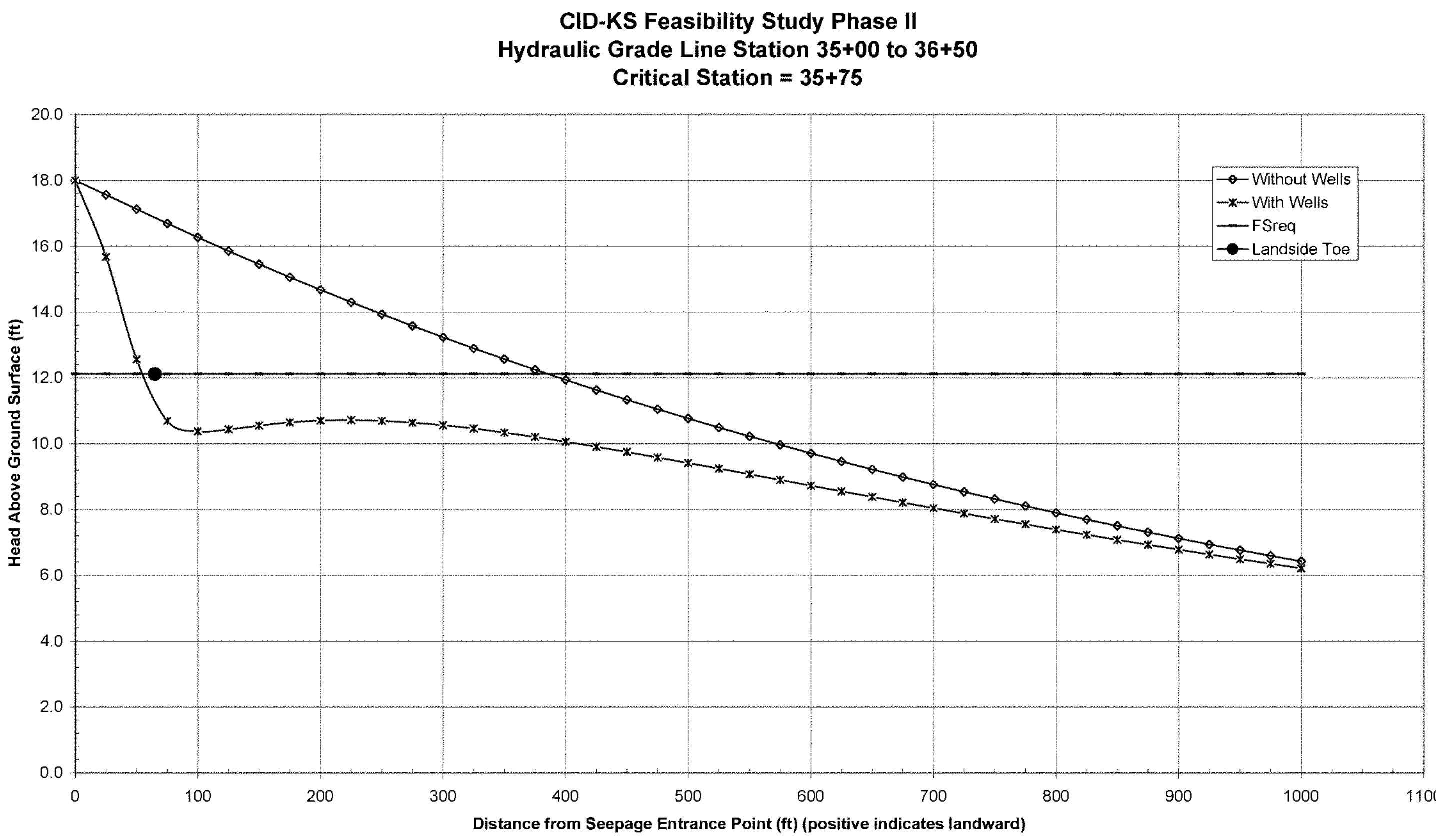
image well locations		
well	x'	y
1	-65	3350
2	-65	3400
3	-65	3450
4	-65	3500
5	-65	3525
6	-65	3550
7	-65	3600
8	-65	3625
9	-75	3637
10	-75	3650
11	-85	3675
12	-100	3700
13	0	0
14	0	0
15	0	0
16	0	0
17	0	0
18	0	0
19	0	0
20	0	0

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

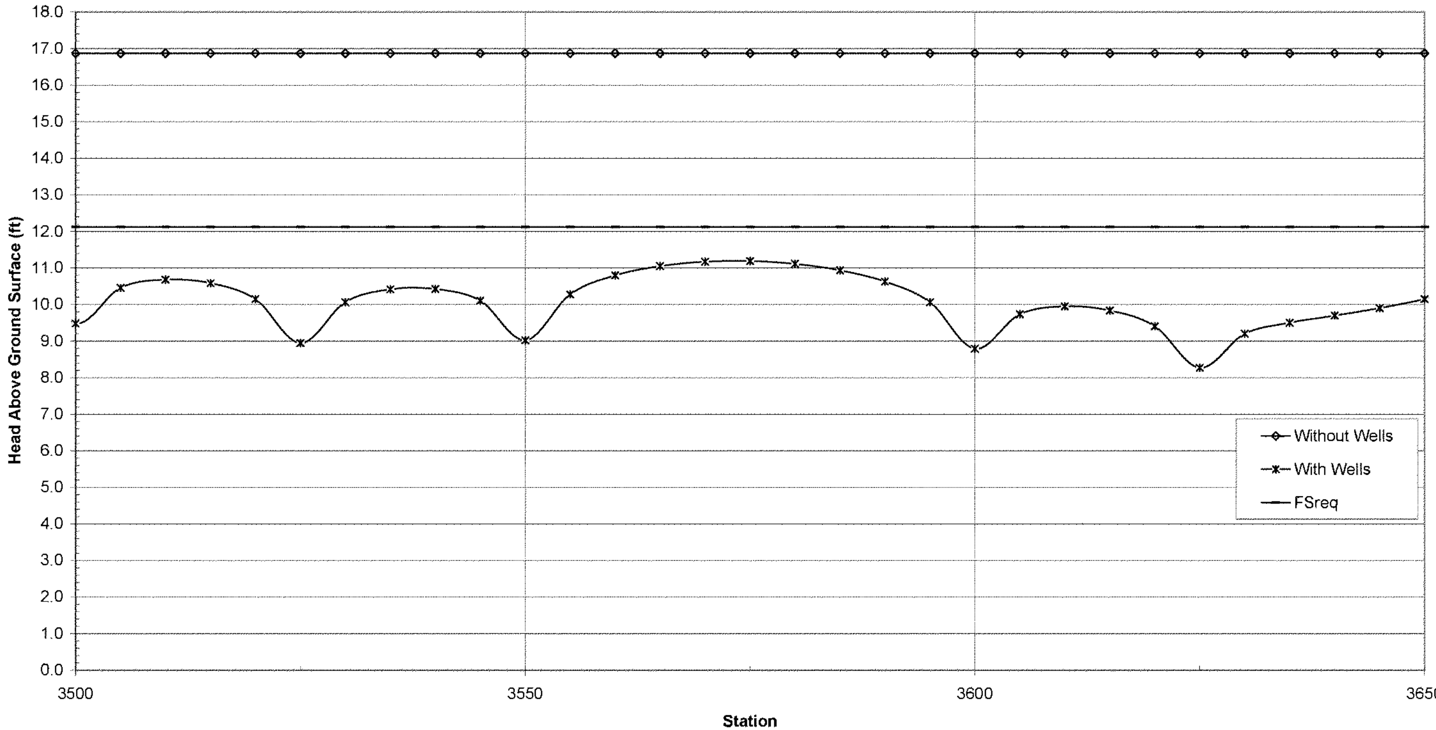
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	3575	18.0	0.0	18.0	12.12	1.00	0.73	1.08
2	25	3575	17.6	2.9	15.7	12.12	1.00	0.68	1.24
3	50	3575	17.1	5.6	12.6	12.12	1.00	0.55	1.54
4	75	3575	16.7	7.0	10.7	12.12	1.00	0.46	1.81
5	100	3575	16.3	6.9	10.4	12.12	1.00	0.45	1.87
6	125	3575	15.9	6.4	10.4	12.12	1.00	0.45	1.86
7	150	3575	15.5	5.9	10.5	12.12	1.00	0.46	1.84
8	175	3575	15.1	5.4	10.6	12.12	1.00	0.46	1.82
9	200	3575	14.7	5.0	10.7	12.12	1.00	0.47	1.81
10	225	3575	14.3	4.6	10.7	12.12	1.00	0.47	1.81
11	250	3575	13.9	4.2	10.7	12.12	1.00	0.46	1.81
12	275	3575	13.6	3.9	10.6	12.12	1.00	0.46	1.82
13	300	3575	13.2	3.7	10.6	12.12	1.00	0.46	1.84
14	325	3575	12.9	3.4	10.5	12.12	1.00	0.45	1.85
15	350	3575	12.6	3.2	10.3	12.12	1.00	0.45	1.88
16	375	3575	12.2	3.0	10.2	12.12	1.00	0.44	1.90
17	400	3575	11.9	2.9	10.1	12.12	1.00	0.44	1.93
18	425	3575	11.6	2.7	9.9	12.12	1.00	0.43	1.96
19	450	3575	11.3	2.6	9.7	12.12	1.00	0.42	1.99
20	475	3575	11.0	2.5	9.6	12.12	1.00	0.42	2.02
21	500	3575	10.8	2.4	9.4	12.12	1.00	0.41	2.06
22	525	3575	10.5	2.2	9.2	12.12	1.00	0.40	2.10
23	550	3575	10.2	2.2	9.1	12.12	1.00	0.39	2.14
24	575	3575	10.0	2.1	8.9	12.12	1.00	0.39	2.18
25	600	3575	9.7	2.0	8.7	12.12	1.00	0.38	2.22
26	625	3575	9.5	1.9	8.6	12.12	1.00	0.37	2.27
27	650	3575	9.2	1.8	8.4	12.12	1.00	0.36	2.31
28	675	3575	9.0	1.8	8.2	12.12	1.00	0.36	2.36
29	700	3575	8.8	1.7	8.0	12.12	1.00	0.35	2.41
30	725	3575	8.5	1.7	7.9	12.12	1.00	0.34	2.46
31	750	3575	8.3	1.6	7.7	12.12	1.00	0.34	2.51
32	775	3575	8.1	1.6	7.6	12.12	1.00	0.33	2.57
33	800	3575	7.9	1.5	7.4	12.12	1.00	0.32	2.62
34	825	3575	7.7	1.5	7.2	12.12	1.00	0.31	2.68
35	850	3575	7.5	1.4	7.1	12.12	1.00	0.31	2.74
36	875	3575	7.3	1.4	6.9	12.12	1.00	0.30	2.80
37	900	3575	7.1	1.3	6.8	12.12	1.00	0.29	2.86
38	925	3575	6.9	1.3	6.6	12.12	1.00	0.29	2.92
39	950	3575	6.8	1.3	6.5	12.12	1.00	0.28	2.99
40	975	3575	6.6	1.2	6.3	12.12	1.00	0.28	3.05
41	1000	3575	6.4	1.2	6.2	12.12	1.00	0.27	3.12

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	65	3490	16.9	6.5	11.4	12.12	1.00	0.50	1.70
2	65	3495	16.9	7.1	10.8	12.12	1.00	0.47	1.79
3	65	3500	16.9	8.4	9.5	12.12	1.00	0.41	2.05
4	65	3505	16.9	7.4	10.5	12.12	1.00	0.45	1.85
5	65	3510	16.9	7.2	10.7	12.12	1.00	0.46	1.81
6	65	3515	16.9	7.3	10.6	12.12	1.00	0.46	1.83
7	65	3520	16.9	7.7	10.1	12.12	1.00	0.44	1.91
8	65	3525	16.9	8.9	8.9	12.12	1.00	0.39	2.17
9	65	3530	16.9	7.8	10.1	12.12	1.00	0.44	1.93
10	65	3535	16.9	7.5	10.4	12.12	1.00	0.45	1.86
11	65	3540	16.9	7.4	10.4	12.12	1.00	0.45	1.86
12	65	3545	16.9	7.8	10.1	12.12	1.00	0.44	1.92
13	65	3550	16.9	8.8	9.0	12.12	1.00	0.39	2.15
14	65	3555	16.9	7.6	10.3	12.12	1.00	0.45	1.89
15	65	3560	16.9	7.1	10.8	12.12	1.00	0.47	1.80
16	65	3565	16.9	6.8	11.1	12.12	1.00	0.48	1.75
17	65	3570	16.9	6.7	11.2	12.12	1.00	0.49	1.74
18	65	3575	16.9	6.7	11.2	12.12	1.00	0.49	1.73
19	65	3580	16.9	6.8	11.1	12.12	1.00	0.48	1.74
20	65	3585	16.9	6.9	10.9	12.12	1.00	0.48	1.77
21	65	3590	16.9	7.2	10.6	12.12	1.00	0.46	1.82
22	65	3595	16.9	7.8	10.1	12.12	1.00	0.44	1.93
23	65	3600	16.9	9.1	8.8	12.12	1.00	0.38	2.20
24	65	3605	16.9	8.1	9.7	12.12	1.00	0.42	1.99
25	65	3610	16.9	7.9	9.9	12.12	1.00	0.43	1.95
26	65	3615	16.9	8.0	9.8	12.12	1.00	0.43	1.97
27	65	3620	16.9	8.5	9.4	12.12	1.00	0.41	2.06
28	65	3625	16.9	9.6	8.3	12.12	1.00	0.36	2.34
29	65	3630	16.9	8.7	9.2	12.12	1.00	0.40	2.11
30	65	3635	16.9	8.4	9.5	12.12	1.00	0.41	2.04
31	65	3640	16.9	8.2	9.7	12.12	1.00	0.42	2.00
32	65	3645	16.9	8.0	9.9	12.12	1.00	0.43	1.96
33	65	3650	16.9	7.7	10.1	12.12	1.00	0.44	1.91
34	65	3655	16.9	7.4	10.5	12.12	1.00	0.46	1.85
35	65	3660	16.9	7.0	10.8	12.12	1.00	0.47	1.79
36	65	3665	16.9	6.7	11.1	12.12	1.00	0.48	1.74
37	65	3670	16.9	6.4	11.4	12.12	1.00	0.50	1.70
38	65	3675	16.9	6.2	11.7	12.12	1.00	0.51	1.66
39	65	3680	16.9	5.9	12.0	12.12	1.00	0.52	1.62
40	65	3685	16.9	5.6	12.2	12.12	1.00	0.53	1.58
41	65	3690	16.9	5.3	12.5	12.12	1.00	0.54	1.55


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 35+00 to 36+50
Critical Station = 35+75
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
Without Wells
With Wells
FSreq
Landside Toe
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 35+00 to 36+50
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	51	ft
h_o =	16.66	ft
TOL =	762	ft elevation
Landside =	745.0	ft elevation
Bottom Blanket =	721	ft elevation
blanket =	24.0	ft
z, Landside Toe =	65	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	12.33	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
1	65	3350	747.0	1.00	0.32	1.00
2	65	3400	747.0	0.91	0.29	1.00
3	65	3450	746.0	0.92	0.29	1.00
4	65	3500	746.0	0.83	0.26	1.00
5	65	3525	745.0	0.85	0.27	1.00
6	65	3550	745.0	0.85	0.27	1.00
7	65	3600	745.0	0.80	0.26	1.00
8	65	3625	745.0	0.75	0.24	1.00
9	75	3637	745.0	0.71	0.23	1.00
10	75	3650	745.0	0.73	0.23	1.00
11	85	3675	746.0	0.73	0.23	1.00
12	100	3700	746.0	0.78	0.25	1.00
13				0.00	0.00	0.00
14				0.00	0.00	0.00
15				0.00	0.00	0.00
16				0.00	0.00	0.00
17				0.00	0.00	0.00
18				0.00	0.00	0.00
19				0.00	0.00	0.00
20				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

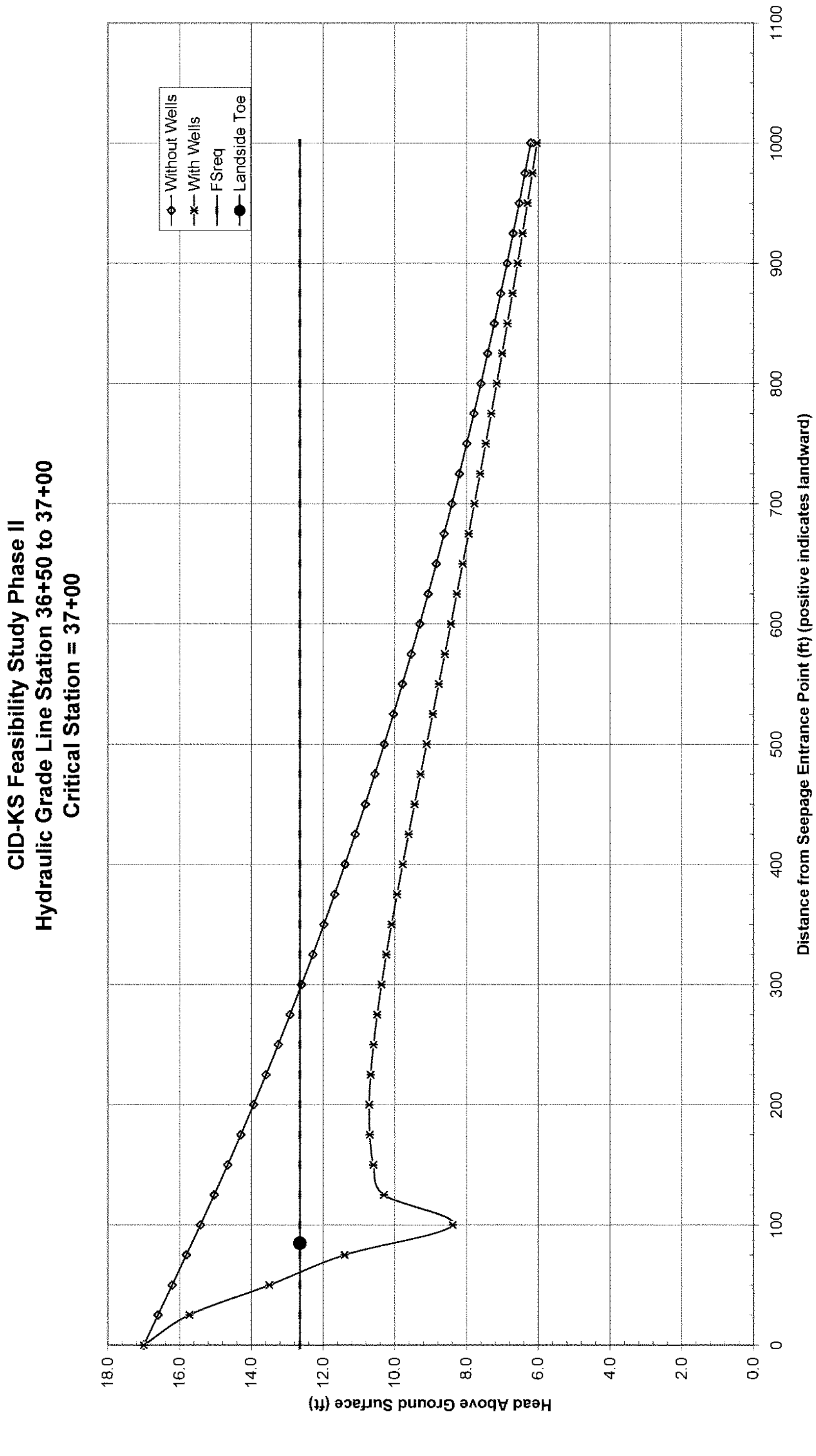
image well locations		
well	x'	y'
1	-65	3350
2	-65	3400
3	-65	3450
4	-65	3500
5	-65	3525
6	-65	3550
7	-65	3600
8	-65	3625
9	-75	3637
10	-75	3650
11	-85	3675
12	-100	3700
13	0	0
14	0	0
15	0	0
16	0	0
17	0	0
18	0	0
19	0	0
20	0	0

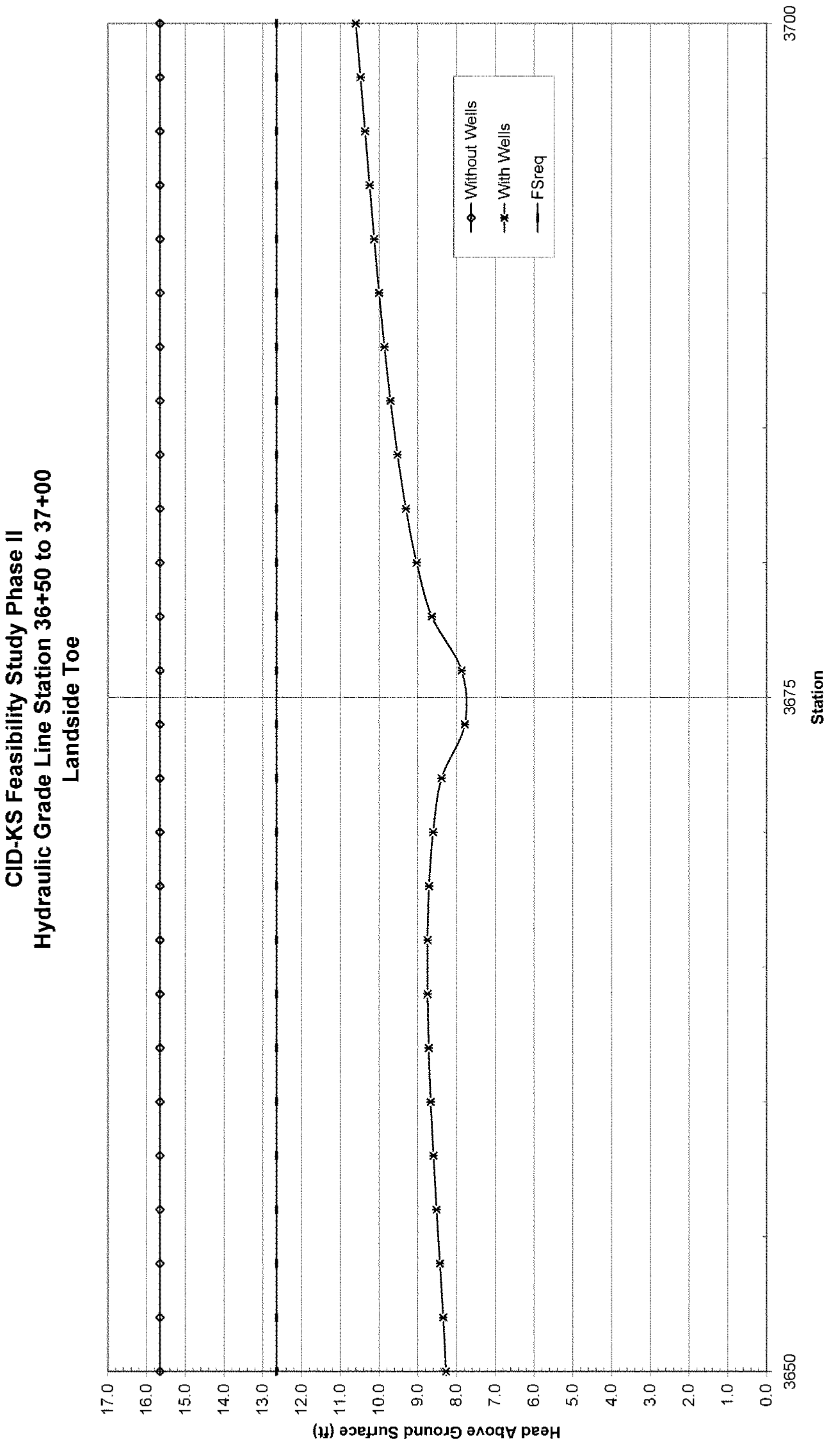
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_e (ft)	H_w (ft)	i	FS_i
1	0	3700	17.0	0.0	17.0	12.64	1.00	0.71	1.19
2	25	3700	16.6	1.9	15.7	12.64	1.00	0.66	1.29
3	50	3700	16.2	3.7	13.5	12.64	1.00	0.56	1.50
4	75	3700	15.8	5.4	11.4	12.64	1.00	0.47	1.77
5	100	3700	15.4	8.0	8.4	12.64	1.00	0.35	2.41
6	125	3700	15.0	5.7	10.3	12.64	1.00	0.43	1.96
7	150	3700	14.7	5.1	10.6	12.64	1.00	0.44	1.91
8	175	3700	14.3	4.6	10.7	12.64	1.00	0.45	1.89
9	200	3700	13.9	4.2	10.7	12.64	1.00	0.45	1.89
10	225	3700	13.6	3.9	10.7	12.64	1.00	0.44	1.90
11	250	3700	13.3	3.7	10.6	12.64	1.00	0.44	1.91
12	275	3700	12.9	3.4	10.5	12.64	1.00	0.44	1.93
13	300	3700	12.6	3.2	10.4	12.64	1.00	0.43	1.95
14	325	3700	12.3	3.0	10.2	12.64	1.00	0.43	1.98
15	350	3700	12.0	2.9	10.1	12.64	1.00	0.42	2.00
16	375	3700	11.7	2.7	9.9	12.64	1.00	0.41	2.04
17	400	3700	11.4	2.6	9.8	12.64	1.00	0.41	2.07
18	425	3700	11.1	2.5	9.6	12.64	1.00	0.40	2.10
19	450	3700	10.8	2.4	9.4	12.64	1.00	0.39	2.14
20	475	3700	10.6	2.3	9.3	12.64	1.00	0.39	2.18
21	500	3700	10.3	2.2	9.1	12.64	1.00	0.38	2.22
22	525	3700	10.0	2.1	8.9	12.64	1.00	0.37	2.26
23	550	3700	9.8	2.0	8.8	12.64	1.00	0.37	2.31
24	575	3700	9.5	1.9	8.6	12.64	1.00	0.36	2.35
25	600	3700	9.3	1.9	8.4	12.64	1.00	0.35	2.40
26	625	3700	9.1	1.8	8.3	12.64	1.00	0.34	2.45
27	650	3700	8.8	1.7	8.1	12.64	1.00	0.34	2.50
28	675	3700	8.6	1.7	7.9	12.64	1.00	0.33	2.55
29	700	3700	8.4	1.6	7.8	12.64	1.00	0.32	2.60
30	725	3700	8.2	1.6	7.6	12.64	1.00	0.32	2.66
31	750	3700	8.0	1.5	7.5	12.64	1.00	0.31	2.71
32	775	3700	7.8	1.5	7.3	12.64	1.00	0.30	2.77
33	800	3700	7.6	1.4	7.2	12.64	1.00	0.30	2.83
34	825	3700	7.4	1.4	7.0	12.64	1.00	0.29	2.89
35	850	3700	7.2	1.4	5.9	12.64	1.00	0.29	2.95
36	875	3700	7.0	1.3	5.7	12.64	1.00	0.28	3.01
37	900	3700	6.9	1.3	5.6	12.64	1.00	0.27	3.08
38	925	3700	6.7	1.3	5.4	12.64	1.00	0.27	3.14
39	950	3700	6.5	1.2	5.3	12.64	1.00	0.26	3.21
40	975	3700	6.4	1.2	5.2	12.64	1.00	0.26	3.28
41	1000	3700	6.2	1.2	6.0	12.64	1.00	0.25	3.35

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_e (ft)	H_w (ft)	i	FS_i
1	65	3640	15.7	8.5	8.1	12.64	1.00	0.34	2.49
2	65	3642	15.7	8.5	8.1	12.64	1.00	0.34	2.49
3	65	3644	15.7	8.5	8.1	12.64	1.00	0.34	2.48
4	65	3646	15.7	8.5	8.2	12.64	1.00	0.34	2.47
5	65	3648	15.7	8.4	8.2	12.64	1.00	0.34	2.46
6	65	3650	15.7	8.4	8.3	12.64	1.00	0.34	2.45
7	65	3652	15.7	8.3	8.3	12.64	1.00	0.35	2.42
8	65	3654	15.7	8.2	8.4	12.64	1.00	0.35	2.40
9	65	3656	15.7	8.1	8.5	12.64	1.00	0.35	2.38
10	65	3658	15.7	8.1	8.6	12.64	1.00	0.36	2.35
11	65	3660	15.7	8.0	8.7	12.64	1.00	0.36	2.33
12	65	3662	15.7	7.9	8.7	12.64	1.00	0.36	2.32
13	65	3664	15.7	7.9	8.7	12.64	1.00	0.36	2.31
14	65	3666	15.7	7.9	8.7	12.64	1.00	0.36	2.31
15	65	3668	15.7	7.9	8.7	12.64	1.00	0.36	2.32
16	65	3670	15.7	8.0	8.6	12.64	1.00	0.36	2.35
17	65	3672	15.7	8.3	8.4	12.64	1.00	0.35	2.41
18	65	3674	15.7	8.9	7.8	12.64	1.00	0.32	2.60
19	65	3676	15.7	8.8	7.9	12.64	1.00	0.33	2.57
20	65	3678	15.7	8.0	8.6	12.64	1.00	0.36	2.34
21	65	3680	15.7	7.6	9.0	12.64	1.00	0.38	2.24
22	65	3682	15.7	7.3	9.3	12.64	1.00	0.39	2.17
23	65	3684	15.7	7.1	9.5	12.64	1.00	0.40	2.12
24	65	3686	15.7	7.0	9.7	12.64	1.00	0.40	2.08
25	65	3688	15.7	6.8	9.9	12.64	1.00	0.41	2.05
26	65	3690	15.7	6.7	10.0	12.64	1.00	0.42	2.02
27	65	3692	15.7	6.5	10.1	12.64	1.00	0.42	2.00
28	65	3694	15.7	6.4	10.2	12.64	1.00	0.43	1.97
29	65	3696	15.7	6.3	10.4	12.64	1.00	0.43	1.95
30	65	3698	15.7	6.2	10.5	12.64	1.00	0.44	1.93
31	65	3700	15.7	6.1	10.6	12.64	1.00	0.44	1.91
32	65	3702	15.7	5.9	10.7	12.64	1.00	0.45	1.89
33	65	3704	15.7	5.8	10.9	12.64	1.00	0.45	1.86
34	65	3706	15.7	5.7	11.0	12.64	1.00	0.46	1.84
35	65	3708	15.7	5.5	11.1	12.64	1.00	0.46	1.82
36	65	3710	15.7	5.4	11.3	12.64	1.00	0.47	1.79
37	65	3712	15.7	5.2	11.4	12.64	1.00	0.48	1.77
38	65	3714	15.7	5.1	11.5	12.64	1.00	0.48	1.75
39	65	3716	15.7	5.0	11.7	12.64	1.00	0.49	1.73
40	65	3718	15.7	4.9	11.8	12.64	1.00	0.49	1.71
41	65	3720	15.7	4.7	11.9	12.64	1.00	0.50	1.70


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 36+50 to 37+00
Critical Station = 37+00
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100
Distance from Seepage Entrance Point (ft) (positive indicates landward)


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 36+50 to 37+00
Landside Toe
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
3650
3675
3700
Station
Without Wells
With Wells
FSreq

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	51	ft
h_o =	14.24	ft
TOL =	762	ft elevation
Landside =	746.0	ft elevation
Bottom Blanket =	721	ft elevation
blanket =	25.0	ft
z, Landside Toe =	125	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	11.98	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
1	65	3350	747.0	0.97	0.31	1.00
2	65	3400	747.0	0.89	0.28	1.00
3	65	3450	746.0	0.89	0.28	1.00
4	65	3500	746.0	0.81	0.26	1.00
5	65	3525	745.0	0.83	0.26	1.00
6	65	3550	745.0	0.82	0.26	1.00
7	65	3600	745.0	0.78	0.25	1.00
8	65	3625	745.0	0.73	0.23	1.00
9	75	3637	745.0	0.69	0.22	1.00
10	75	3650	745.0	0.71	0.23	1.00
11	85	3675	746.0	0.70	0.22	1.00
12	100	3700	746.0	0.76	0.24	1.00
13				0.00	0.00	0.00
14				0.00	0.00	0.00
15				0.00	0.00	0.00
16				0.00	0.00	0.00
17				0.00	0.00	0.00
18				0.00	0.00	0.00
19				0.00	0.00	0.00
20				0.00	0.00	0.00
						1.00

<--Input H_w AVG after any changes are made to well parameters

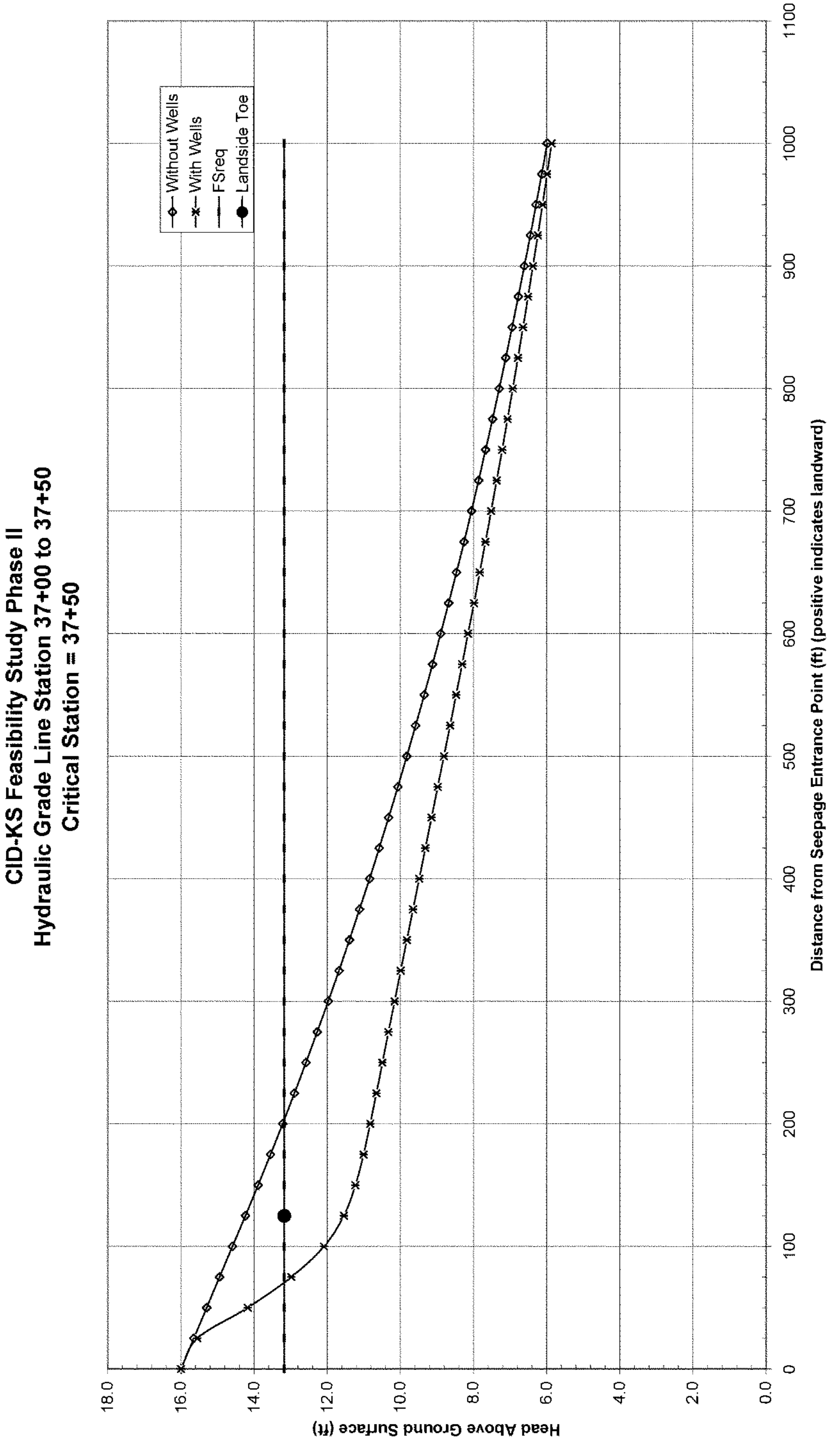
image well locations		
well	x'	y
1	-65	3350
2	-65	3400
3	-65	3450
4	-65	3500
5	-65	3525
6	-65	3550
7	-65	3600
8	-65	3625
9	-75	3637
10	-75	3650
11	-85	3675
12	-100	3700
13	0	0
14	0	0
15	0	0
16	0	0
17	0	0
18	0	0
19	0	0
20	0	0

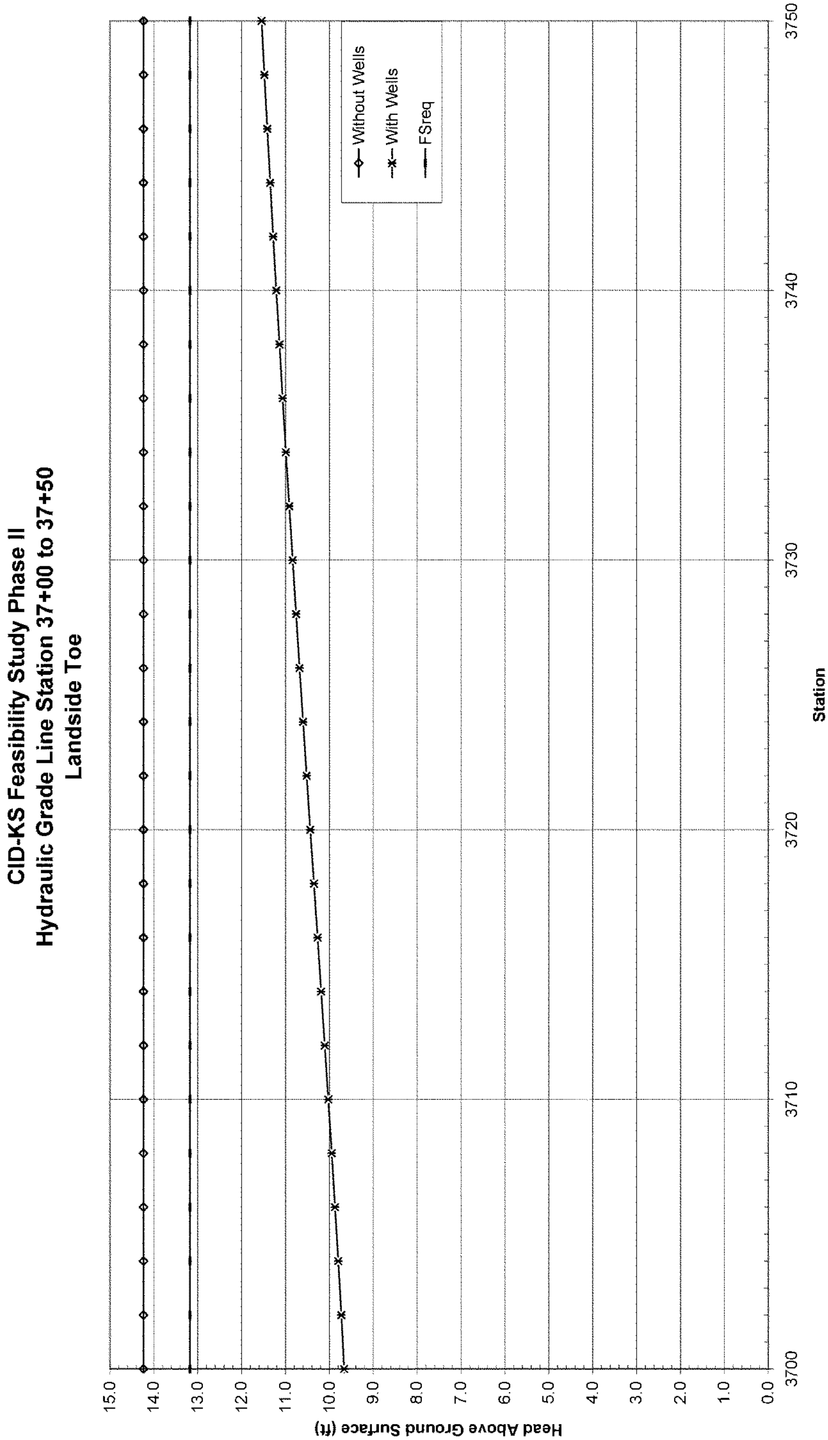
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_e (ft)	H_w (ft)	i	FS_i
1	0	3750	16.0	0.0	16.0	13.17	1.00	0.64	1.32
2	25	3750	15.6	1.1	15.5	13.17	1.00	0.62	1.36
3	50	3750	15.3	2.1	14.2	13.17	1.00	0.57	1.49
4	75	3750	14.9	3.0	13.0	13.17	1.00	0.52	1.62
5	100	3750	14.6	3.5	12.1	13.17	1.00	0.48	1.74
6	125	3750	14.2	3.7	11.5	13.17	1.00	0.46	1.83
7	150	3750	13.9	3.7	11.2	13.17	1.00	0.45	1.88
8	175	3750	13.6	3.5	11.0	13.17	1.00	0.44	1.91
9	200	3750	13.2	3.4	10.8	13.17	1.00	0.43	1.95
10	225	3750	12.9	3.2	10.7	13.17	1.00	0.43	1.96
11	250	3750	12.6	3.1	10.5	13.17	1.00	0.42	2.01
12	275	3750	12.3	2.9	10.3	13.17	1.00	0.41	2.04
13	300	3750	12.0	2.8	10.2	13.17	1.00	0.41	2.07
14	325	3750	11.7	2.7	10.0	13.17	1.00	0.40	2.11
15	350	3750	11.4	2.6	9.8	13.17	1.00	0.39	2.15
16	375	3750	11.1	2.5	9.7	13.17	1.00	0.39	2.18
17	400	3750	10.8	2.4	9.5	13.17	1.00	0.38	2.22
18	425	3750	10.6	2.3	9.3	13.17	1.00	0.37	2.26
19	450	3750	10.3	2.2	9.1	13.17	1.00	0.37	2.30
20	475	3750	10.1	2.1	9.0	13.17	1.00	0.36	2.35
21	500	3750	9.8	2.0	8.8	13.17	1.00	0.35	2.39
22	525	3750	9.6	1.9	8.6	13.17	1.00	0.35	2.44
23	550	3750	9.3	1.9	8.5	13.17	1.00	0.34	2.49
24	575	3750	9.1	1.8	8.3	13.17	1.00	0.33	2.54
25	600	3750	8.9	1.7	8.1	13.17	1.00	0.33	2.59
26	625	3750	8.7	1.7	8.0	13.17	1.00	0.32	2.64
27	650	3750	8.5	1.6	7.8	13.17	1.00	0.31	2.69
28	675	3750	8.3	1.6	7.7	13.17	1.00	0.31	2.75
29	700	3750	8.1	1.5	7.5	13.17	1.00	0.30	2.80
30	725	3750	7.9	1.5	7.4	13.17	1.00	0.29	2.86
31	750	3750	7.7	1.5	7.2	13.17	1.00	0.29	2.92
32	775	3750	7.5	1.4	7.1	13.17	1.00	0.28	2.98
33	800	3750	7.3	1.4	6.9	13.17	1.00	0.28	3.04
34	825	3750	7.1	1.3	6.8	13.17	1.00	0.27	3.11
35	850	3750	6.9	1.3	6.6	13.17	1.00	0.27	3.17
36	875	3750	6.8	1.3	6.5	13.17	1.00	0.26	3.24
37	900	3750	6.6	1.2	6.4	13.17	1.00	0.25	3.31
38	925	3750	6.4	1.2	6.2	13.17	1.00	0.25	3.38
39	950	3750	6.3	1.2	6.1	13.17	1.00	0.24	3.45
40	975	3750	6.1	1.1	6.0	13.17	1.00	0.24	3.52
41	1000	3750	6.0	1.1	5.9	13.17	1.00	0.23	3.59

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_e (ft)	H_w (ft)	i	FS_i
1	125	3690	14.2	5.8	9.4	13.17	1.00	0.38	2.24
2	125	3692	14.2	5.8	9.4	13.17	1.00	0.38	2.23
3	125	3694	14.2	5.7	9.5	13.17	1.00	0.38	2.22
4	125	3696	14.2	5.7	9.5	13.17	1.00	0.38	2.21
5	125	3698	14.2	5.6	9.6	13.17	1.00	0.38	2.19
6	125	3700	14.2	5.6	9.7	13.17	1.00	0.39	2.18
7	125	3702	14.2	5.5	9.7	13.17	1.00	0.39	2.17
8	125	3704	14.2	5.4	9.8	13.17	1.00	0.39	2.15
9	125	3706	14.2	5.4	9.9	13.17	1.00	0.39	2.13
10	125	3708	14.2	5.3	9.9	13.17	1.00	0.40	2.12
11	125	3710	14.2	5.2	10.0	13.17	1.00	0.40	2.10
12	125	3712	14.2	5.1	10.1	13.17	1.00	0.40	2.09
13	125	3714	14.2	5.1	10.2	13.17	1.00	0.41	2.07
14	125	3716	14.2	5.0	10.3	13.17	1.00	0.41	2.05
15	125	3718	14.2	4.9	10.4	13.17	1.00	0.41	2.04
16	125	3720	14.2	4.8	10.4	13.17	1.00	0.42	2.02
17	125	3722	14.2	4.7	10.5	13.17	1.00	0.42	2.00
18	125	3724	14.2	4.6	10.6	13.17	1.00	0.42	1.99
19	125	3726	14.2	4.6	10.7	13.17	1.00	0.43	1.97
20	125	3728	14.2	4.5	10.8	13.17	1.00	0.43	1.96
21	125	3730	14.2	4.4	10.8	13.17	1.00	0.43	1.94
22	125	3732	14.2	4.3	10.9	13.17	1.00	0.44	1.93
23	125	3734	14.2	4.2	11.0	13.17	1.00	0.44	1.92
24	125	3736	14.2	4.2	11.1	13.17	1.00	0.44	1.90
25	125	3738	14.2	4.1	11.1	13.17	1.00	0.45	1.89
26	125	3740	14.2	4.0	11.2	13.17	1.00	0.45	1.88
27	125	3742	14.2	4.0	11.3	13.17	1.00	0.45	1.87
28	125	3744	14.2	3.9	11.3	13.17	1.00	0.45	1.86
29	125	3746	14.2	3.8	11.4	13.17	1.00	0.46	1.85
30	125	3748	14.2	3.8	11.5	13.17	1.00	0.46	1.84
31	125	3750	14.2	3.7	11.5	13.17	1.00	0.46	1.83
32	125	3752	14.2	3.6	11.6	13.17	1.00	0.46	1.82
33	125	3754	14.2	3.6	11.7	13.17	1.00	0.47	1.81
34	125	3756	14.2	3.5	11.7	13.17	1.00	0.47	1.80
35	125	3758	14.2	3.5	11.8	13.17	1.00	0.47	1.79
36	125	3760	14.2	3.4	11.8	13.17	1.00	0.47	1.78
37	125	3762	14.2	3.3	11.9	13.17	1.00	0.48	1.77
38	125	3764	14.2	3.3	12.0	13.17	1.00	0.48	1.76
39	125	3766	14.2	3.2	12.0	13.17	1.00	0.48	1.75
40	125	3768	14.2	3.2	12.1	13.17	1.00	0.48	1.75
41	125	3770	14.2	3.1	12.1	13.17	1.00	0.48	1.74


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 37+00 to 37+50
Critical Station = 37+50
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 37+00 to 37+50
Landside Toe
Head Above Ground Surface (ft)
Station
Without Wells
With Wells
FSreq
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
3700
3710
3720
3730
3740
3750

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	51	ft
h_b =	12.56	ft
TOL =	762	ft elevation
Landside =	747.0	ft elevation
Bottom Blanket =	721	ft elevation
blanket =	26.0	ft
z, Landside Toe =	200	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	11.42	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
1	65	3350	747.0	0.92	0.29	1.00
2	65	3400	747.0	0.84	0.27	1.00
3	65	3450	746.0	0.85	0.27	1.00
4	65	3500	746.0	0.77	0.24	1.00
5	65	3525	745.0	0.79	0.25	1.00
6	65	3550	745.0	0.79	0.25	1.00
7	65	3600	745.0	0.75	0.24	1.00
8	65	3625	745.0	0.70	0.22	1.00
9	75	3637	745.0	0.66	0.21	1.00
10	75	3650	745.0	0.68	0.22	1.00
11	85	3675	746.0	0.67	0.21	1.00
12	100	3700	746.0	0.72	0.23	1.00
13				0.00	0.00	0.00
14				0.00	0.00	0.00
15				0.00	0.00	0.00
16				0.00	0.00	0.00
17				0.00	0.00	0.00
18				0.00	0.00	0.00
19				0.00	0.00	0.00
20				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

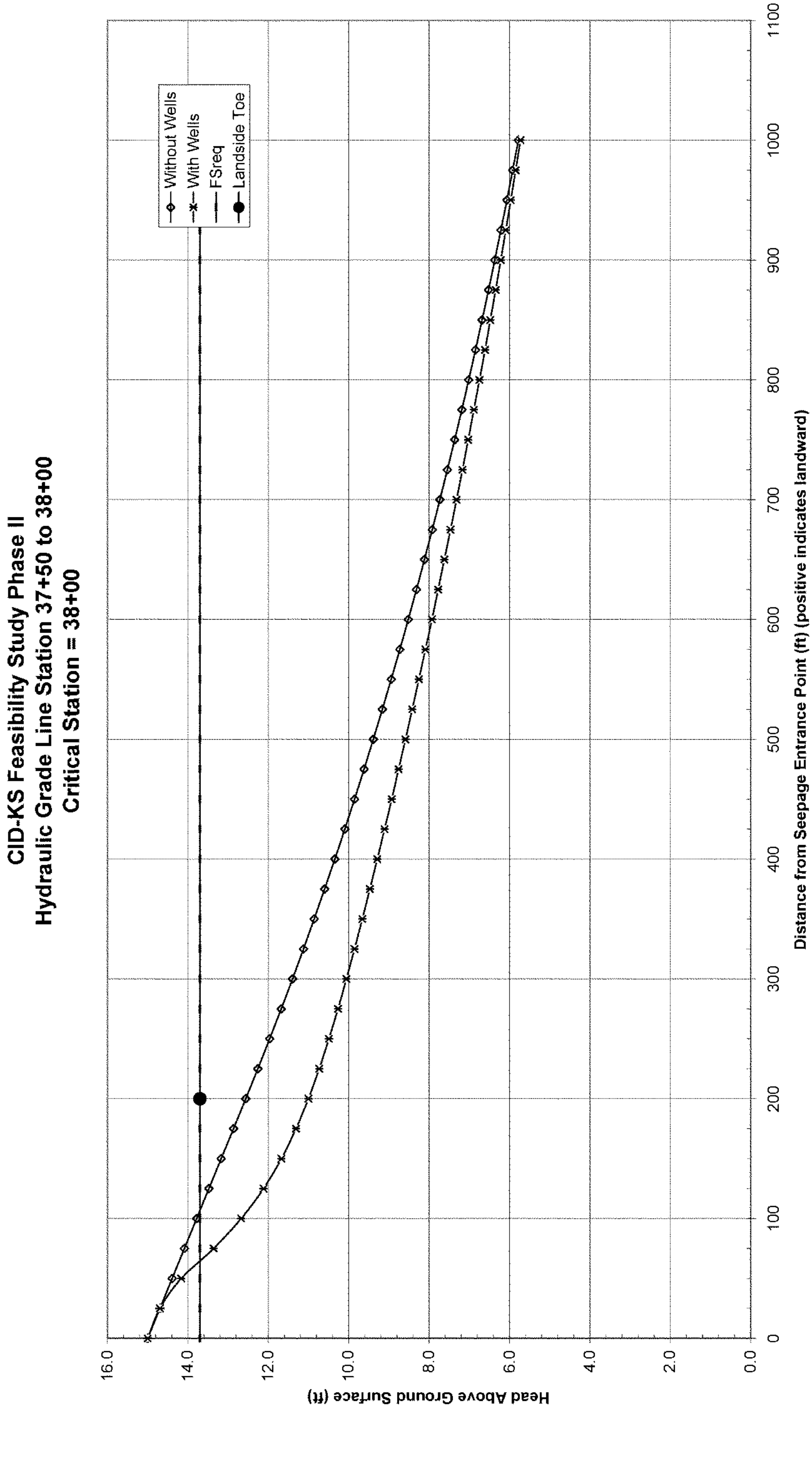
image well locations		
well	x'	y
1	-65	3350
2	-65	3400
3	-65	3450
4	-65	3500
5	-65	3525
6	-65	3550
7	-65	3600
8	-65	3625
9	-75	3637
10	-75	3650
11	-85	3675
12	-100	3700
13	0	0
14	0	0
15	0	0
16	0	0
17	0	0
18	0	0
19	0	0
20	0	0

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	3800	15.0	0.0	15.0	13.70	1.00	0.58	1.46
2	25	3800	14.7	0.6	14.7	13.70	1.00	0.57	1.49
3	50	3800	14.4	1.2	14.2	13.70	1.00	0.54	1.55
4	75	3800	14.1	1.7	13.4	13.70	1.00	0.51	1.64
5	100	3800	13.8	2.1	12.7	13.70	1.00	0.49	1.73
6	125	3800	13.5	2.4	12.1	13.70	1.00	0.47	1.81
7	150	3800	13.2	2.5	11.7	13.70	1.00	0.45	1.88
8	175	3800	12.9	2.6	11.3	13.70	1.00	0.43	1.94
9	200	3800	12.6	2.6	11.0	13.70	1.00	0.42	1.99
10	225	3800	12.3	2.5	10.7	13.70	1.00	0.41	2.04
11	250	3800	12.0	2.5	10.5	13.70	1.00	0.40	2.09
12	275	3800	11.7	2.4	10.3	13.70	1.00	0.39	2.13
13	300	3800	11.4	2.3	10.1	13.70	1.00	0.39	2.18
14	325	3800	11.1	2.3	9.9	13.70	1.00	0.38	2.22
15	350	3800	10.9	2.2	9.7	13.70	1.00	0.37	2.27
16	375	3800	10.6	2.1	9.5	13.70	1.00	0.36	2.31
17	400	3800	10.3	2.1	9.3	13.70	1.00	0.36	2.36
18	425	3800	10.1	2.0	9.1	13.70	1.00	0.35	2.41
19	450	3800	9.9	1.9	8.9	13.70	1.00	0.34	2.45
20	475	3800	9.6	1.9	8.8	13.70	1.00	0.34	2.50
21	500	3800	9.4	1.8	8.6	13.70	1.00	0.33	2.55
22	525	3800	9.2	1.7	8.4	13.70	1.00	0.32	2.60
23	550	3800	8.9	1.7	8.3	13.70	1.00	0.32	2.66
24	575	3800	8.7	1.6	8.1	13.70	1.00	0.31	2.71
25	600	3800	8.5	1.6	7.9	13.70	1.00	0.31	2.76
26	625	3800	8.3	1.5	7.8	13.70	1.00	0.30	2.82
27	650	3800	8.1	1.5	7.6	13.70	1.00	0.29	2.88
28	675	3800	7.9	1.5	7.5	13.70	1.00	0.29	2.94
29	700	3800	7.7	1.4	7.3	13.70	1.00	0.28	3.00
30	725	3800	7.5	1.4	7.2	13.70	1.00	0.28	3.06
31	750	3800	7.4	1.3	7.0	13.70	1.00	0.27	3.12
32	775	3800	7.2	1.3	6.9	13.70	1.00	0.26	3.18
33	800	3800	7.0	1.3	6.7	13.70	1.00	0.26	3.25
34	825	3800	6.8	1.2	6.6	13.70	1.00	0.25	3.32
35	850	3800	6.7	1.2	6.5	13.70	1.00	0.25	3.38
36	875	3800	6.5	1.2	6.3	13.70	1.00	0.24	3.45
37	900	3800	6.4	1.1	6.2	13.70	1.00	0.24	3.53
38	925	3800	6.2	1.1	6.1	13.70	1.00	0.23	3.60
39	950	3800	6.1	1.1	6.0	13.70	1.00	0.23	3.67
40	975	3800	5.9	1.1	5.8	13.70	1.00	0.22	3.75
41	1000	3800	5.8	1.0	5.7	13.70	1.00	0.22	3.83

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	200	3740	12.6	3.4	10.2	13.70	1.00	0.39	2.15
2	200	3742	12.6	3.4	10.2	13.70	1.00	0.39	2.15
3	200	3744	12.6	3.3	10.2	13.70	1.00	0.39	2.14
4	200	3746	12.6	3.3	10.3	13.70	1.00	0.39	2.13
5	200	3748	12.6	3.3	10.3	13.70	1.00	0.40	2.13
6	200	3750	12.6	3.2	10.3	13.70	1.00	0.40	2.12
7	200	3752	12.6	3.2	10.4	13.70	1.00	0.40	2.12
8	200	3754	12.6	3.2	10.4	13.70	1.00	0.40	2.11
9	200	3756	12.6	3.2	10.4	13.70	1.00	0.40	2.11
10	200	3758	12.6	3.1	10.4	13.70	1.00	0.40	2.10
11	200	3760	12.6	3.1	10.5	13.70	1.00	0.40	2.09
12	200	3762	12.6	3.1	10.5	13.70	1.00	0.40	2.09
13	200	3764	12.6	3.0	10.5	13.70	1.00	0.40	2.08
14	200	3766	12.6	3.0	10.6	13.70	1.00	0.41	2.08
15	200	3768	12.6	3.0	10.6	13.70	1.00	0.41	2.07
16	200	3770	12.6	3.0	10.6	13.70	1.00	0.41	2.07
17	200	3772	12.6	2.9	10.6	13.70	1.00	0.41	2.06
18	200	3774	12.6	2.9	10.7	13.70	1.00	0.41	2.06
19	200	3776	12.6	2.9	10.7	13.70	1.00	0.41	2.05
20	200	3778	12.6	2.8	10.7	13.70	1.00	0.41	2.05
21	200	3780	12.6	2.8	10.7	13.70	1.00	0.41	2.04
22	200	3782	12.6	2.8	10.8	13.70	1.00	0.41	2.04
23	200	3784	12.6	2.8	10.8	13.70	1.00	0.42	2.03
24	200	3786	12.6	2.7	10.8	13.70	1.00	0.42	2.03
25	200	3788	12.6	2.7	10.8	13.70	1.00	0.42	2.02
26	200	3790	12.6	2.7	10.9	13.70	1.00	0.42	2.02
27	200	3792	12.6	2.7	10.9	13.70	1.00	0.42	2.01
28	200	3794	12.6	2.6	10.9	13.70	1.00	0.42	2.01
29	200	3796	12.6	2.6	11.0	13.70	1.00	0.42	2.00
30	200	3798	12.6	2.6	11.0	13.70	1.00	0.42	2.00
31	200	3800	12.6	2.6	11.0	13.70	1.00	0.42	1.99
32	200	3802	12.6	2.5	11.0	13.70	1.00	0.42	1.99
33	200	3804	12.6	2.5	11.1	13.70	1.00	0.43	1.98
34	200	3806	12.6	2.5	11.1	13.70	1.00	0.43	1.98
35	200	3808	12.6	2.5	11.1	13.70	1.00	0.43	1.97
36	200	3810	12.6	2.4	11.1	13.70	1.00	0.43	1.97
37	200	3812	12.6	2.4	11.1	13.70	1.00	0.43	1.97
38	200	3814	12.6	2.4	11.2	13.70	1.00	0.43	1.96
39	200	3816	12.6	2.4	11.2	13.70	1.00	0.43	1.96
40	200	3818	12.6	2.3	11.2	13.70	1.00	0.43	1.95
41	200	3820	12.6	2.3	11.2	13.70	1.00	0.43	1.95


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 37+50 to 38+00
Critical Station = 38+00
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100
Distance from Seepage Entrance Point (ft) (positive indicates landward)

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 37+50 to 38+00
Landside Toe

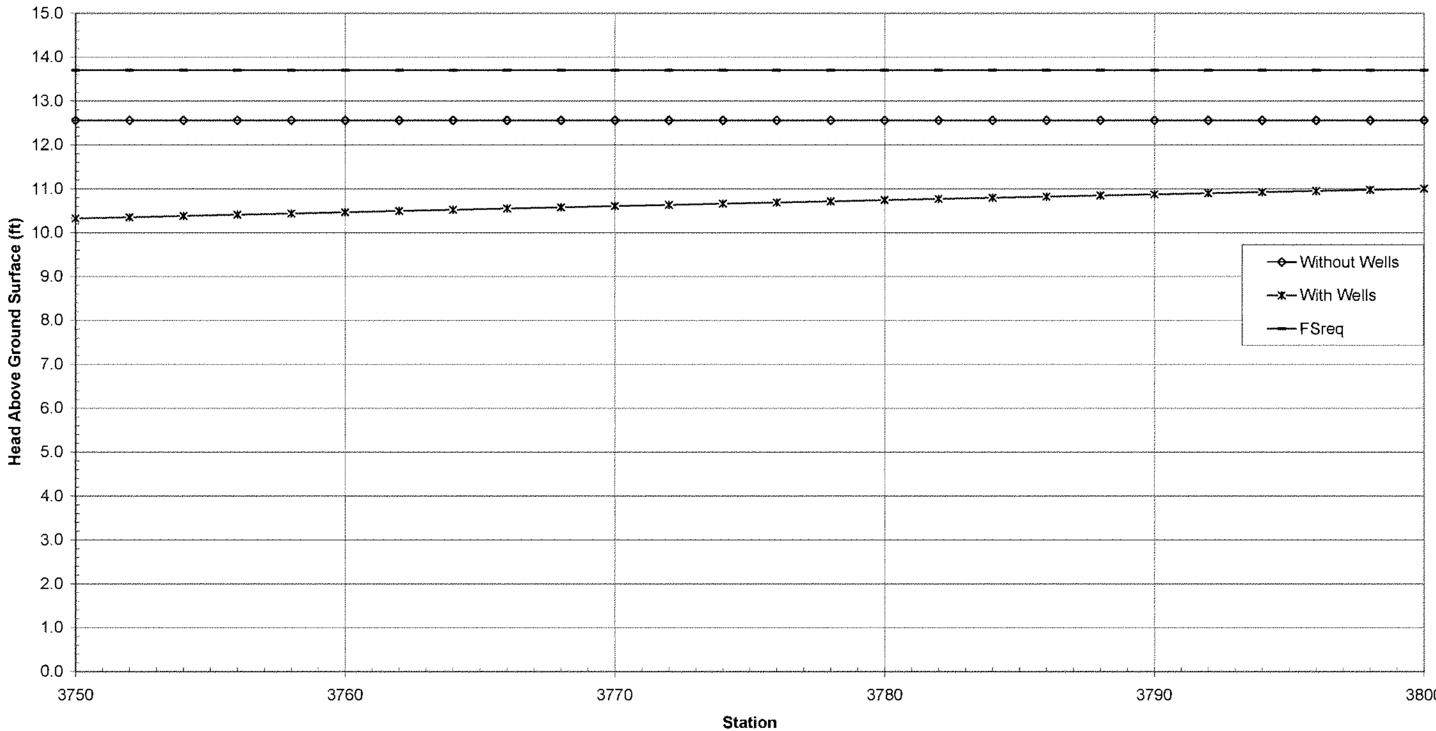


Exhibit A-4.16
Area Fill Design - Station 51+00 to 79+00

CIOKS Levee Kansas City Phase 2 Feasibility
Monday, May 05, 2008 Revised:
Levee Foundation Information - EXISTING CONDITIONS, Water to Top of Levee

[illegible]

IMPERVIOUS AREA FILL DESIGN

[illegible]

IMPERVIOUS AREA FILL DESIGN - BERM TOE CHECK

[illegible]

PERVIOUS AREA FILL DESIGN

[illegible]

EXHIBIT A-4.17

Relief Well Design – Station 51+00 to 79+00

CID-KS Levee Unit 500+3 Feasibility Study

UNDERSEEPAGE ANALYSIS WITHOUT WELLS

Levee Foundation Information - 500+3

Blanket Unit Weight = 115.0 pcf (saturated)

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

N500+3 Relief Well Summary Station 51+00 to 79+00

Well	Status	Distance From Seepage Entrance (ft)	Station	Discharge Elevation (ft)	0.8*Qw (cfs)	Qw (cfs)
8	Existing	152.00	79+30	739.00	1.71	2.13
9	Existing	152.71	80+50	739.99	1.68	2.09
10	Existing	162.00	81+80	740.26	0.81	1.02
11	Existing	154.68	83+30	741.02	1.05	1.31
12	Existing	163.48	85+70	743.50	1.14	1.43
13	Existing	169.23	87+76	744.99	0.89	1.11
14	Existing	161.81	89+80	747.01	1.06	1.33
15	Existing	160.00	92+00	748.01	1.05	1.31
16	Existing	158.66	94+36	750.02	1.00	1.25
17	Existing	160.00	97+00	750.94	0.98	1.22
				Total	**11.36**	**14.20**

18	Proposed	80.00	57+50	750.00	1.08	1.36
19	Proposed	80.00	59+00	750.00	1.06	1.32
20	Proposed	80.00	60+50	750.00	1.08	1.36
21	Proposed	90.00	62+50	748.00	1.09	1.37
22	Proposed	90.00	63+00	748.00	1.00	1.25
23	Proposed	90.00	63+50	748.00	0.96	1.20
24	Proposed	90.00	64+00	748.00	0.94	1.17
25	Proposed	90.00	64+50	748.00	0.93	1.16
26	Proposed	90.00	65+00	748.00	0.92	1.15
27	Proposed	90.00	65+50	748.00	0.92	1.15
28	Proposed	90.00	66+00	748.00	0.92	1.15
29	Proposed	90.00	66+50	748.00	0.93	1.16
30	Proposed	90.00	67+00	748.00	0.97	1.21
31	Proposed	90.00	67+50	748.00	0.96	1.20
32	Proposed	90.00	68+00	748.00	1.00	1.25
33	Proposed	90.00	68+50	748.00	1.09	1.37
34	Proposed	100.00	70+50	747.00	1.21	1.51
35	Proposed	100.00	72+50	747.00	1.20	1.50
36	Proposed	100.00	72+75	746.00	0.79	0.99
37	Proposed	100.00	73+00	746.00	0.75	0.94
38	Proposed	100.00	73+25	746.00	0.73	0.91
39	Proposed	100.00	73+50	746.00	0.71	0.89
40	Proposed	100.00	73+75	746.00	0.71	0.88
41	Proposed	100.00	74+00	746.00	0.71	0.89
42	Proposed	100.00	74+25	746.00	0.72	0.90
43	Proposed	100.00	74+50	746.00	0.82	1.03
44	Proposed	100.00	74+70	746.00	0.77	0.96
45	Proposed	100.00	74+80	746.00	0.78	0.97
46	Proposed	150.00	76+50	742.00	0.91	1.14
47	Proposed	150.00	77+00	742.00	0.86	1.07
48	Proposed	150.00	77+50	742.00	0.83	1.04
49	Proposed	150.00	78+00	742.00	0.82	1.02
50	Proposed	150.00	78+50	742.00	0.82	1.02
51	Proposed	150.00	79+50	742.00	0.95	1.19

52	Proposed	150.00	80+50	742.00	0.71	0.89
53	Proposed	150.00	81+40	742.00	0.78	0.98
54	Proposed	150.00	82+00	742.00	0.76	0.95
55	Proposed	150.00	82+25	742.00	0.81	1.01
56	Proposed	150.00	82+50	740.00	0.63	0.79
57	Proposed	150.00	82+75	740.00	0.91	1.13
58	Proposed	150.00	83+25	740.00	0.74	0.92
59	Proposed	150.00	83+50	740.00	0.74	0.92
60	Proposed	150.00	83+90	740.00	0.74	0.93
61	Proposed	150.00	84+30	740.00	0.72	0.90
62	Proposed	150.00	84+60	740.00	0.71	0.89
63	Proposed	150.00	84+90	740.00	0.71	0.88
64	Proposed	150.00	85+20	740.00	0.71	0.88
65	Proposed	150.00	85+50	740.00	0.75	0.94
66	Proposed	150.00	86+25	742.00	0.99	1.23
67	Proposed	150.00	86+75	742.00	0.93	1.17
68	Proposed	150.00	87+30	742.00	0.89	1.11
69	Proposed	150.00	87+80	742.00	1.02	1.28
70	Proposed	150.00	88+20	742.00	1.04	1.30
71	Proposed	150.00	89+50	745.00	0.89	1.11
72	Proposed	150.00	90+75	745.00	0.93	1.16
73	Proposed	150.00	91+50	745.00	1.27	1.58
				Total	**49.32**	**61.65**

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	50	ft
h_p =	11.96	ft
TOL =	763.2	ft elevation
Landside =	750.0	ft elevation
Bottom Blanket =	720	ft elevation
blanket =	30.0	ft
z, Landside Toe =	80	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.64	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	0.00	cfs

real well locations							image well locations		
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)	well	x'	y'
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0
				0.00	0.00	0.00	0	0	0

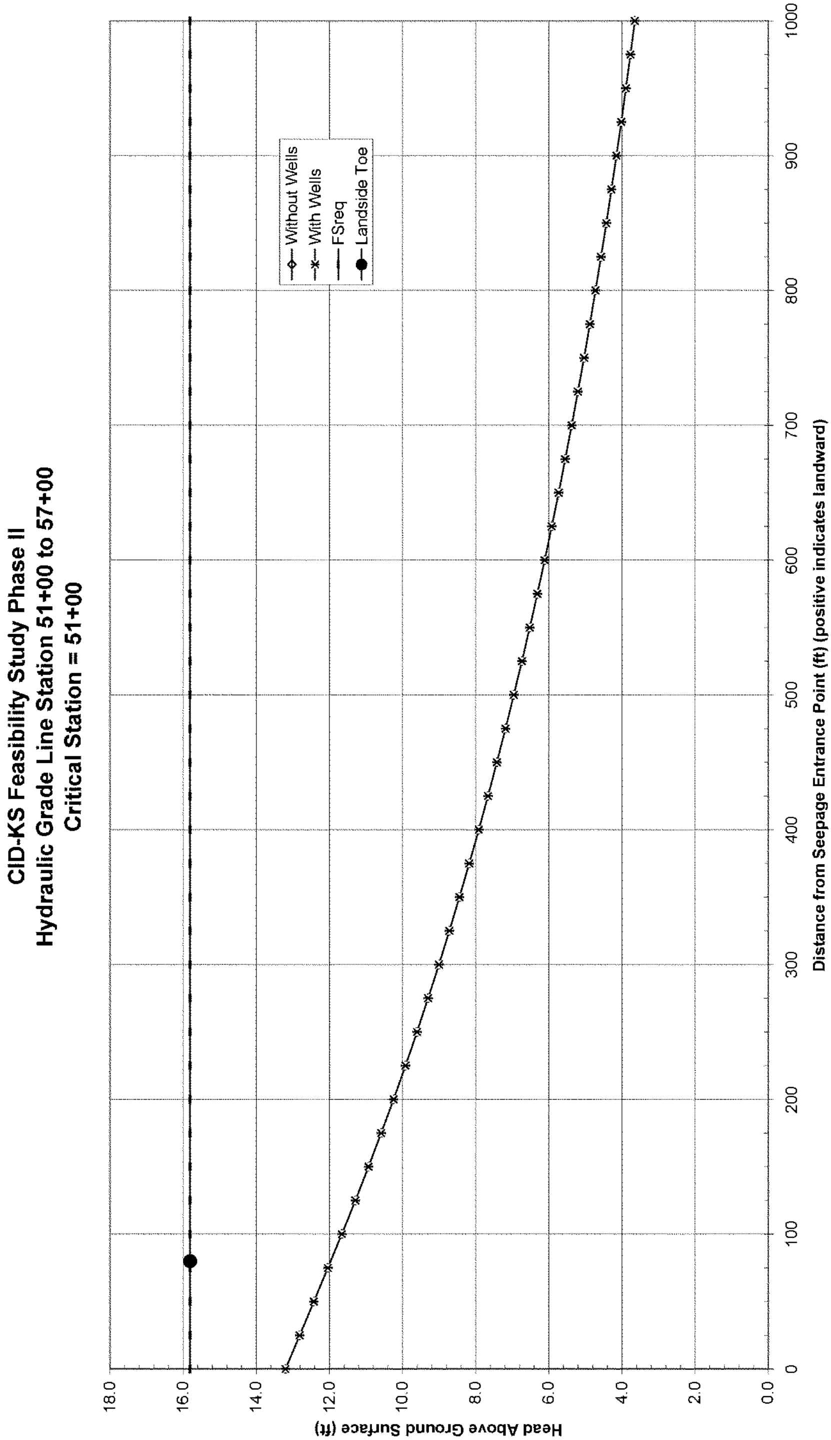
1.00 **<---Input H_w AVG after any changes are made to well parameters**

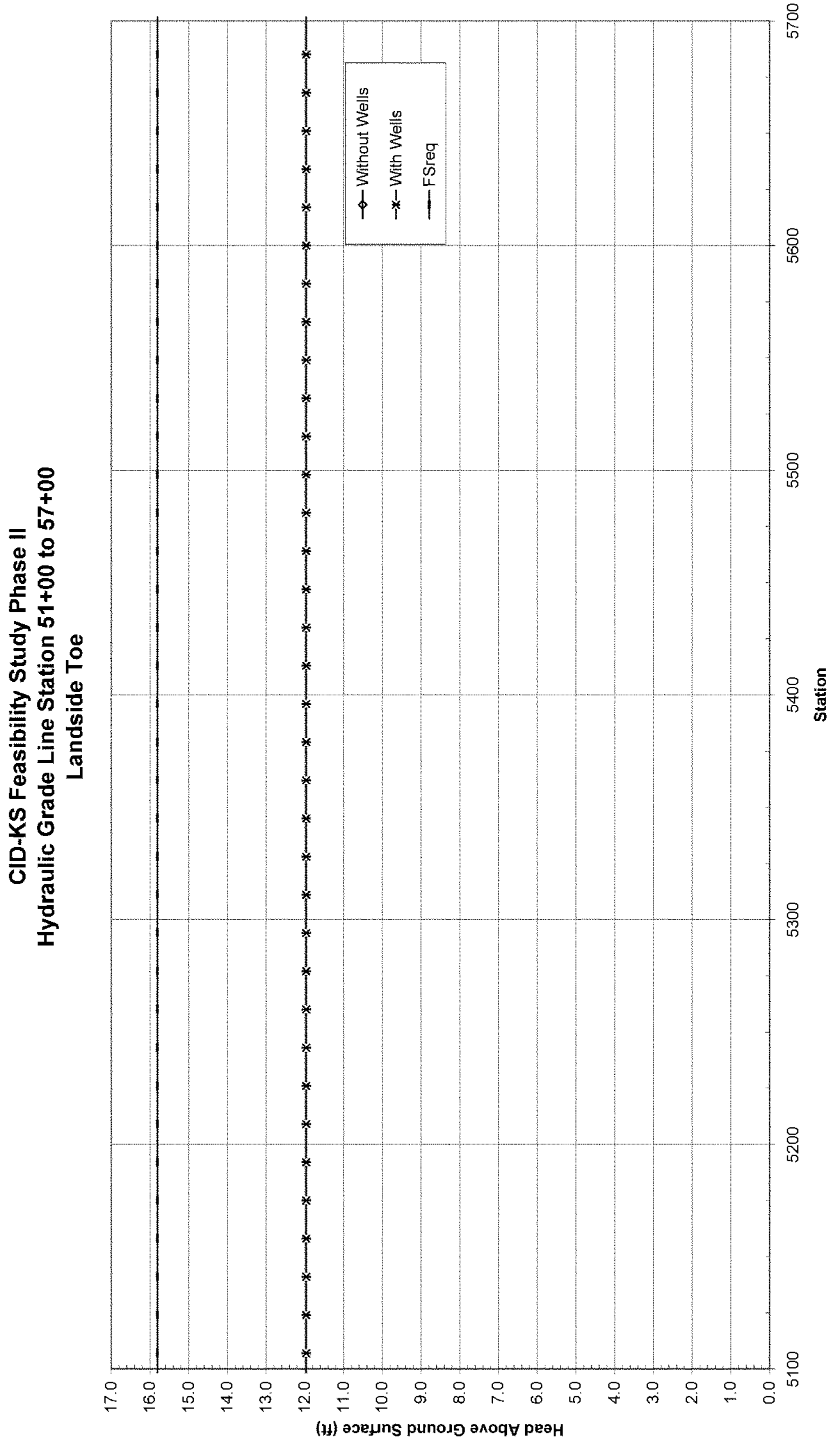
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	5100	13.2	0.0	13.2	15.81	1.00	0.44	1.92
2	25	5100	12.9	0.0	12.8	15.81	1.00	0.43	1.97
3	50	5100	12.4	0.0	12.4	15.81	1.00	0.41	2.03
4	75	5100	12.0	0.0	12.0	15.81	1.00	0.40	2.10
5	100	5100	11.7	0.0	11.7	15.81	1.00	0.39	2.17
6	125	5100	11.3	0.0	11.3	15.81	1.00	0.38	2.24
7	150	5100	10.9	0.0	10.9	15.81	1.00	0.36	2.31
8	175	5100	10.6	0.0	10.6	15.81	1.00	0.35	2.39
9	200	5100	10.2	0.0	10.2	15.81	1.00	0.34	2.47
10	225	5100	9.9	0.0	9.9	15.81	1.00	0.33	2.55
11	250	5100	9.6	0.0	9.6	15.81	1.00	0.32	2.63
12	275	5100	9.3	0.0	9.3	15.81	1.00	0.31	2.72
13	300	5100	9.0	0.0	9.0	15.81	1.00	0.30	2.81
14	325	5100	8.7	0.0	8.7	15.81	1.00	0.29	2.90
15	350	5100	8.4	0.0	8.4	15.81	1.00	0.28	3.00
16	375	5100	8.2	0.0	8.2	15.81	1.00	0.27	3.09
17	400	5100	7.9	0.0	7.9	15.81	1.00	0.26	3.19
18	425	5100	7.7	0.0	7.7	15.81	1.00	0.26	3.30
19	450	5100	7.4	0.0	7.4	15.81	1.00	0.25	3.41
20	475	5100	7.2	0.0	7.2	15.81	1.00	0.24	3.52
21	500	5100	7.0	0.0	7.0	15.81	1.00	0.23	3.64
22	525	5100	6.7	0.0	6.7	15.81	1.00	0.22	3.75
23	550	5100	6.5	0.0	6.5	15.81	1.00	0.22	3.88
24	575	5100	6.3	0.0	6.3	15.81	1.00	0.21	4.00
25	600	5100	6.1	0.0	6.1	15.81	1.00	0.20	4.14
26	625	5100	5.9	0.0	5.9	15.81	1.00	0.20	4.27
27	650	5100	5.7	0.0	5.7	15.81	1.00	0.19	4.41
28	675	5100	5.5	0.0	5.5	15.81	1.00	0.18	4.56
29	700	5100	5.4	0.0	5.4	15.81	1.00	0.18	4.71
30	725	5100	5.2	0.0	5.2	15.81	1.00	0.17	4.86
31	750	5100	5.0	0.0	5.0	15.81	1.00	0.17	5.02
32	775	5100	4.9	0.0	4.9	15.81	1.00	0.16	5.18
33	800	5100	4.7	0.0	4.7	15.81	1.00	0.16	5.35
34	825	5100	4.6	0.0	4.6	15.81	1.00	0.15	5.53
35	850	5100	4.4	0.0	4.4	15.81	1.00	0.15	5.71
36	875	5100	4.3	0.0	4.3	15.81	1.00	0.14	5.90
37	900	5100	4.2	0.0	4.2	15.81	1.00	0.14	6.09
38	925	5100	4.0	0.0	4.0	15.81	1.00	0.13	6.29
39	950	5100	3.9	0.0	3.9	15.81	1.00	0.13	6.50
40	975	5100	3.8	0.0	3.8	15.81	1.00	0.13	6.71
41	1000	5100	3.6	0.0	3.6	15.81	1.00	0.12	6.93

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	80	5090	12.0	0.0	12.0	15.81	1.00	0.40	2.11
2	80	5107	12.0	0.0	12.0	15.81	1.00	0.40	2.11
3	80	5124	12.0	0.0	12.0	15.81	1.00	0.40	2.11
4	80	5141	12.0	0.0	12.0	15.81	1.00	0.40	2.11
5	80	5158	12.0	0.0	12.0	15.81	1.00	0.40	2.11
6	80	5175	12.0	0.0	12.0	15.81	1.00	0.40	2.11
7	80	5192	12.0	0.0	12.0	15.81	1.00	0.40	2.11
8	80	5209	12.0	0.0	12.0	15.81	1.00	0.40	2.11
9	80	5226	12.0	0.0	12.0	15.81	1.00	0.40	2.11
10	80	5243	12.0	0.0	12.0	15.81	1.00	0.40	2.11
11	80	5260	12.0	0.0	12.0	15.81	1.00	0.40	2.11
12	80	5277	12.0	0.0	12.0	15.81	1.00	0.40	2.11
13	80	5294	12.0	0.0	12.0	15.81	1.00	0.40	2.11
14	80	5311	12.0	0.0	12.0	15.81	1.00	0.40	2.11
15	80	5328	12.0	0.0	12.0	15.81	1.00	0.40	2.11
16	80	5345	12.0	0.0	12.0	15.81	1.00	0.40	2.11
17	80	5362	12.0	0.0	12.0	15.81	1.00	0.40	2.11
18	80	5379	12.0	0.0	12.0	15.81	1.00	0.40	2.11
19	80	5396	12.0	0.0	12.0	15.81	1.00	0.40	2.11
20	80	5413	12.0	0.0	12.0	15.81	1.00	0.40	2.11
21	80	5430	12.0	0.0	12.0	15.81	1.00	0.40	2.11
22	80	5447	12.0	0.0	12.0	15.81	1.00	0.40	2.11
23	80	5464	12.0	0.0	12.0	15.81	1.00	0.40	2.11
24	80	5481	12.0	0.0	12.0	15.81	1.00	0.40	2.11
25	80	5498	12.0	0.0	12.0	15.81	1.00	0.40	2.11
26	80	5515	12.0	0.0	12.0	15.81	1.00	0.40	2.11
27	80	5532	12.0	0.0	12.0	15.81	1.00	0.40	2.11
28	80	5549	12.0	0.0	12.0	15.81	1.00	0.40	2.11
29	80	5566	12.0	0.0	12.0	15.81	1.00	0.40	2.11
30	80	5583	12.0	0.0	12.0	15.81	1.00	0.40	2.11
31	80	5600	12.0	0.0	12.0	15.81	1.00	0.40	2.11
32	80	5617	12.0	0.0	12.0	15.81	1.00	0.40	2.11
33	80	5634	12.0	0.0	12.0	15.81	1.00	0.40	2.11
34	80	5651	12.0	0.0	12.0	15.81	1.00	0.40	2.11
35	80	5668	12.0	0.0	12.0	15.81	1.00	0.40	2.11
36	80	5685	12.0	0.0	12.0	15.81	1.00	0.40	2.11
37	80	5702	12.0	0.0	12.0	15.81	1.00	0.40	2.11
38	80	5719	12.0	0.0	12.0	15.81	1.00	0.40	2.11
39	80	5736	12.0	0.0	12.0	15.81	1.00	0.40	2.11
40	80	5753	12.0	0.0	12.0	15.81	1.00	0.40	2.11
41	80	5770	12.0	0.0	12.0	15.81	1.00	0.40	2.11


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 51+00 to 57+00
Critical Station = 51+00
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 51+00 to 57+00
Landside Toe
Without Wells
With Wells
FSreq
Head Above Ground Surface (ft)
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0
17.0
Station
5100
5200
5300
5400
5500
5600
5700

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	52	ft
h_0 =	12.28	ft
TOL =	763.7	ft elevation
Landside =	750.0	ft elevation
Bottom Blanket =	722	ft elevation
blanket =	23.0	ft
z, Landside Toe =	80	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.64	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	4.03	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	V_w (ft/s)	H_w (ft)
18	80	5750	750.0	1.08	0.35	1.01
19	80	5900	750.0	1.06	0.34	1.00
20	80	6050	750.0	1.08	0.35	1.01
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00

image well locations		
well	x'	y'
18	-80	5750
19	-80	5900
20	-80	6050
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

1.00 **<---Input H_w AVG after any changes are made to well parameters**

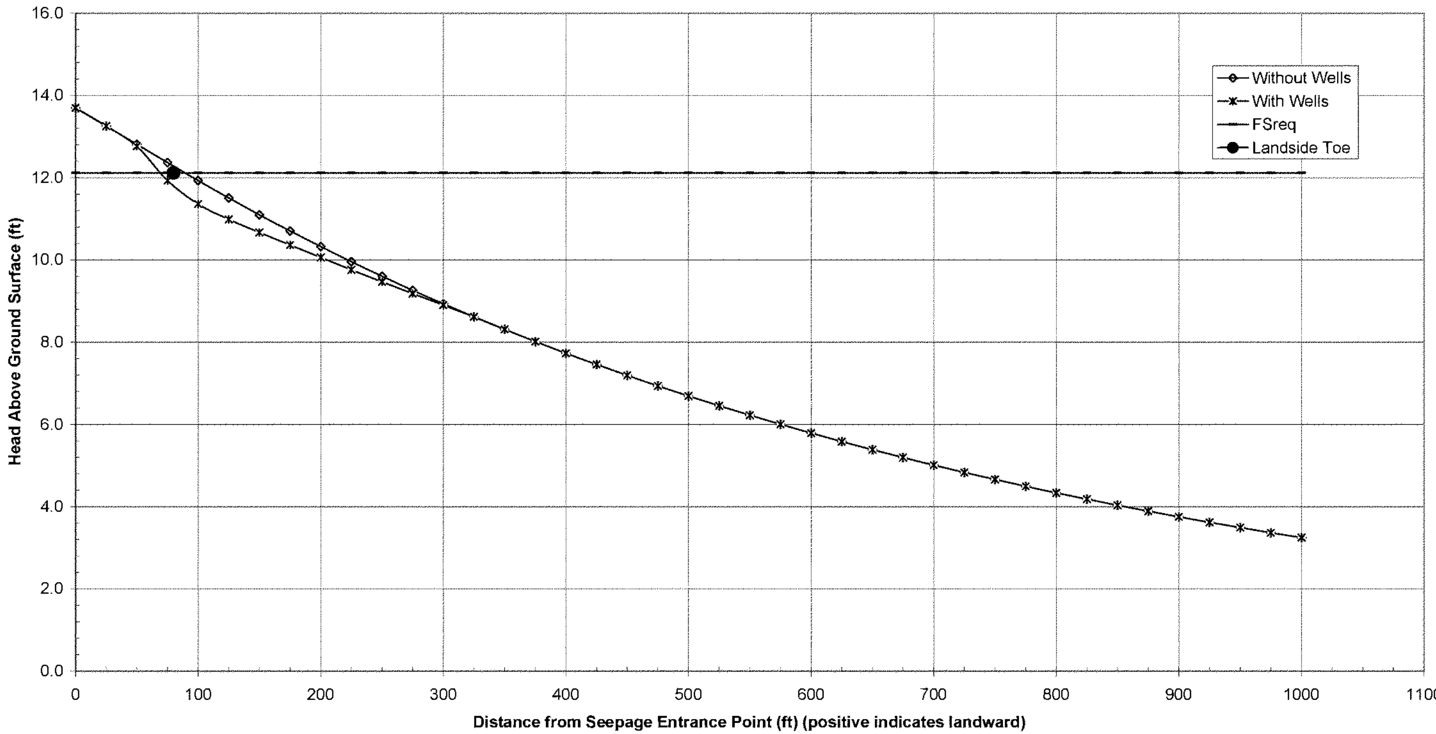
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

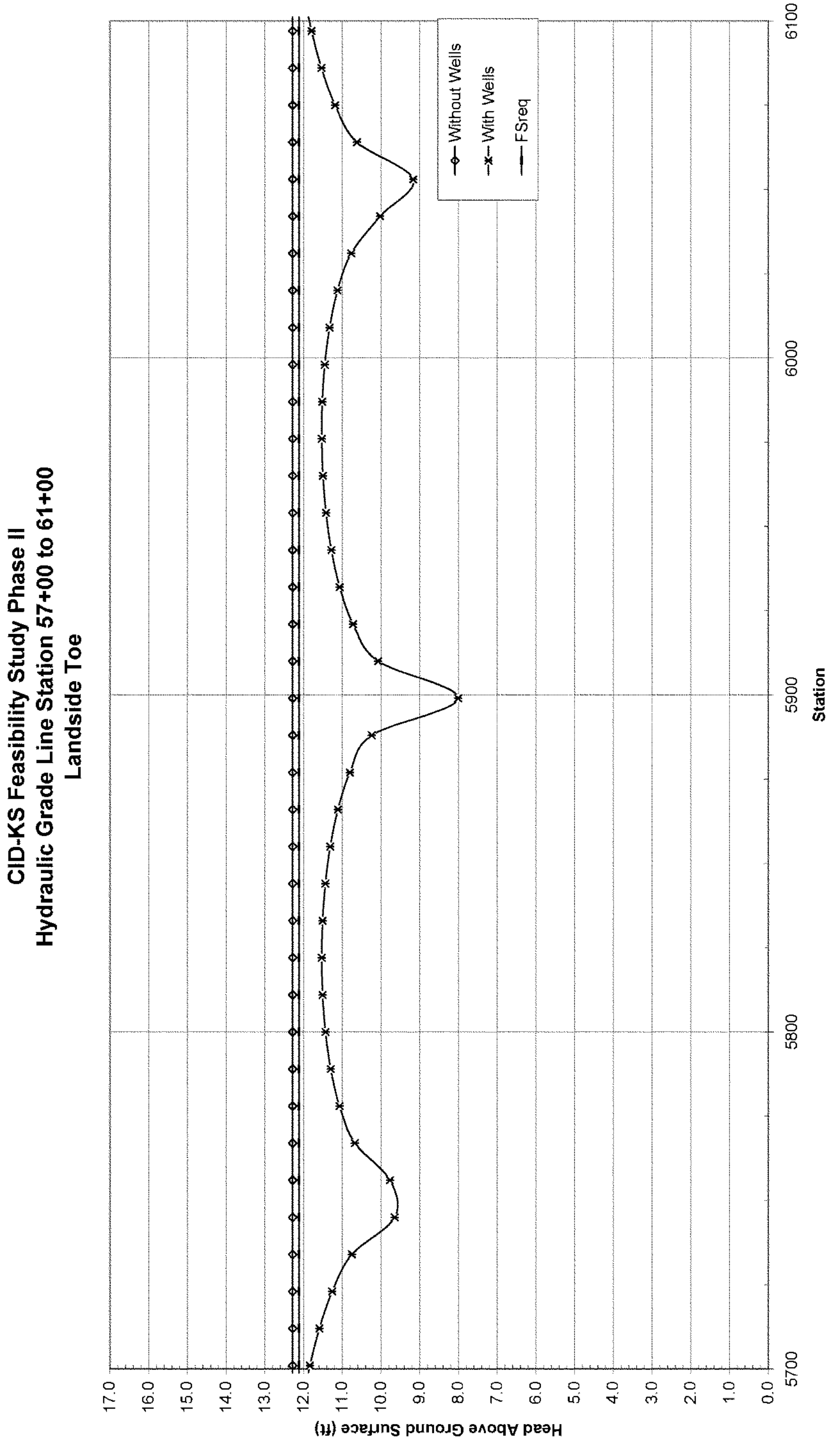
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	6097	13.7	0.0	13.7	12.12	1.00	0.60	1.42
2	25	6097	13.3	0.5	13.3	12.12	1.00	0.58	1.46
3	50	6097	12.8	1.0	12.8	12.12	1.00	0.56	1.52
4	75	6097	12.4	1.4	11.9	12.12	1.00	0.52	1.62
5	100	6097	11.9	1.6	11.4	12.12	1.00	0.49	1.71
6	125	6097	11.5	1.5	11.0	12.12	1.00	0.48	1.76
7	150	6097	11.1	1.4	10.7	12.12	1.00	0.46	1.82
8	175	6097	10.7	1.3	10.4	12.12	1.00	0.45	1.87
9	200	6097	10.3	1.3	10.1	12.12	1.00	0.44	1.93
10	225	6097	10.0	1.2	9.8	12.12	1.00	0.42	1.99
11	250	6097	9.6	1.1	9.5	12.12	1.00	0.41	2.05
12	275	6097	9.3	1.1	9.2	12.12	1.00	0.40	2.11
13	300	6097	8.9	1.0	8.9	12.12	1.00	0.39	2.18
14	325	6097	8.6	1.0	8.6	12.12	1.00	0.37	2.25
15	350	6097	8.3	0.9	8.3	12.12	1.00	0.36	2.33
16	375	6097	8.0	0.9	8.0	12.12	1.00	0.35	2.42
17	400	6097	7.7	0.9	7.7	12.12	1.00	0.34	2.51
18	425	6097	7.5	0.8	7.5	12.12	1.00	0.32	2.60
19	450	6097	7.2	0.8	7.2	12.12	1.00	0.31	2.70
20	475	6097	6.9	0.8	6.9	12.12	1.00	0.30	2.79
21	500	6097	6.7	0.7	6.7	12.12	1.00	0.29	2.90
22	525	6097	6.5	0.7	6.5	12.12	1.00	0.28	3.00
23	550	6097	6.2	0.7	6.2	12.12	1.00	0.27	3.11
24	575	6097	6.0	0.7	6.0	12.12	1.00	0.26	3.23
25	600	6097	5.8	0.6	5.8	12.12	1.00	0.25	3.35
26	625	6097	5.6	0.6	5.6	12.12	1.00	0.24	3.47
27	650	6097	5.4	0.6	5.4	12.12	1.00	0.23	3.60
28	675	6097	5.2	0.6	5.2	12.12	1.00	0.23	3.73
29	700	6097	5.0	0.6	5.0	12.12	1.00	0.22	3.87
30	725	6097	4.8	0.6	4.8	12.12	1.00	0.21	4.01
31	750	6097	4.7	0.5	4.7	12.12	1.00	0.20	4.16
32	775	6097	4.5	0.5	4.5	12.12	1.00	0.20	4.31
33	800	6097	4.3	0.5	4.3	12.12	1.00	0.19	4.47
34	825	6097	4.2	0.5	4.2	12.12	1.00	0.18	4.64
35	850	6097	4.0	0.5	4.0	12.12	1.00	0.18	4.81
36	875	6097	3.9	0.5	3.9	12.12	1.00	0.17	4.98
37	900	6097	3.8	0.5	3.8	12.12	1.00	0.16	5.17
38	925	6097	3.6	0.4	3.6	12.12	1.00	0.16	5.36
39	950	6097	3.5	0.4	3.5	12.12	1.00	0.15	5.55
40	975	6097	3.4	0.4	3.4	12.12	1.00	0.15	5.76
41	1000	6097	3.2	0.4	3.2	12.12	1.00	0.14	5.97

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	80	5690	12.3	1.3	12.0	12.12	1.00	0.52	1.61
2	80	5701	12.3	1.4	11.8	12.12	1.00	0.51	1.64
3	80	5712	12.3	1.7	11.6	12.12	1.00	0.50	1.67
4	80	5723	12.3	2.0	11.3	12.12	1.00	0.49	1.72
5	80	5734	12.3	2.5	10.8	12.12	1.00	0.47	1.80
6	80	5745	12.3	3.6	9.6	12.12	1.00	0.42	2.01
7	80	5756	12.3	3.5	9.8	12.12	1.00	0.42	1.98
8	80	5767	12.3	2.6	10.7	12.12	1.00	0.46	1.82
9	80	5778	12.3	2.2	11.1	12.12	1.00	0.48	1.75
10	80	5789	12.3	2.0	11.3	12.12	1.00	0.49	1.72
11	80	5800	12.3	1.8	11.4	12.12	1.00	0.50	1.70
12	80	5811	12.3	1.8	11.5	12.12	1.00	0.50	1.68
13	80	5822	12.3	1.7	11.5	12.12	1.00	0.50	1.68
14	80	5833	12.3	1.8	11.5	12.12	1.00	0.50	1.68
15	80	5844	12.3	1.8	11.4	12.12	1.00	0.50	1.70
16	80	5855	12.3	2.0	11.3	12.12	1.00	0.49	1.71
17	80	5866	12.3	2.2	11.1	12.12	1.00	0.48	1.74
18	80	5877	12.3	2.5	10.8	12.12	1.00	0.47	1.80
19	80	5888	12.3	3.0	10.2	12.12	1.00	0.45	1.89
20	80	5899	12.3	5.3	8.0	12.12	1.00	0.35	2.42
21	80	5910	12.3	3.2	10.1	12.12	1.00	0.44	1.92
22	80	5921	12.3	2.6	10.7	12.12	1.00	0.47	1.81
23	80	5932	12.3	2.2	11.1	12.12	1.00	0.48	1.75
24	80	5943	12.3	2.0	11.3	12.12	1.00	0.49	1.72
25	80	5954	12.3	1.9	11.4	12.12	1.00	0.50	1.70
26	80	5965	12.3	1.8	11.5	12.12	1.00	0.50	1.69
27	80	5976	12.3	1.7	11.5	12.12	1.00	0.50	1.68
28	80	5987	12.3	1.8	11.5	12.12	1.00	0.50	1.68
29	80	5998	12.3	1.8	11.5	12.12	1.00	0.50	1.69
30	80	6009	12.3	2.0	11.3	12.12	1.00	0.49	1.71
31	80	6020	12.3	2.2	11.1	12.12	1.00	0.48	1.74
32	80	6031	12.3	2.5	10.8	12.12	1.00	0.47	1.80
33	80	6042	12.3	3.3	10.0	12.12	1.00	0.44	1.93
34	80	6053	12.3	4.1	9.2	12.12	1.00	0.40	2.11
35	80	6064	12.3	2.7	10.6	12.12	1.00	0.46	1.83
36	80	6075	12.3	2.1	11.2	12.12	1.00	0.49	1.73
37	80	6086	12.3	1.7	11.5	12.12	1.00	0.50	1.68
38	80	6097	12.3	1.5	11.8	12.12	1.00	0.51	1.64
39	80	6108	12.3	1.3	12.0	12.12	1.00	0.52	1.62
40	80	6119	12.3	1.1	12.2	12.12	1.00	0.53	1.60
41	80	6130	12.3	1.0	12.3	12.12	1.00	0.53	1.58

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 57+00 to 61+00
Critical Station = 60+97




CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 57+00 to 61+00
Landside Toe
Head Above Ground Surface (ft)
Station
Without Wells
With Wells
FSreq
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
5700
5800
5900
6000
6100

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	55	ft
h_o =	12.55	ft
TOL =	763.9	ft elevation
Landside =	750.0	ft elevation
Bottom Blanket =	725	ft elevation
blanket =	25.0	ft
z, Landside Toe =	80	ft
t_w =	1	ft

γ_{sd} =	115	pcf
i_c =	0.84	
FS_{eq} =	1.6	
Efficiency =	0.8	
Total Flow =	0.00	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00

1.00 **<---Input H_w AVG after any changes are made to well parameters**

image well locations		
well	x'	y'
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

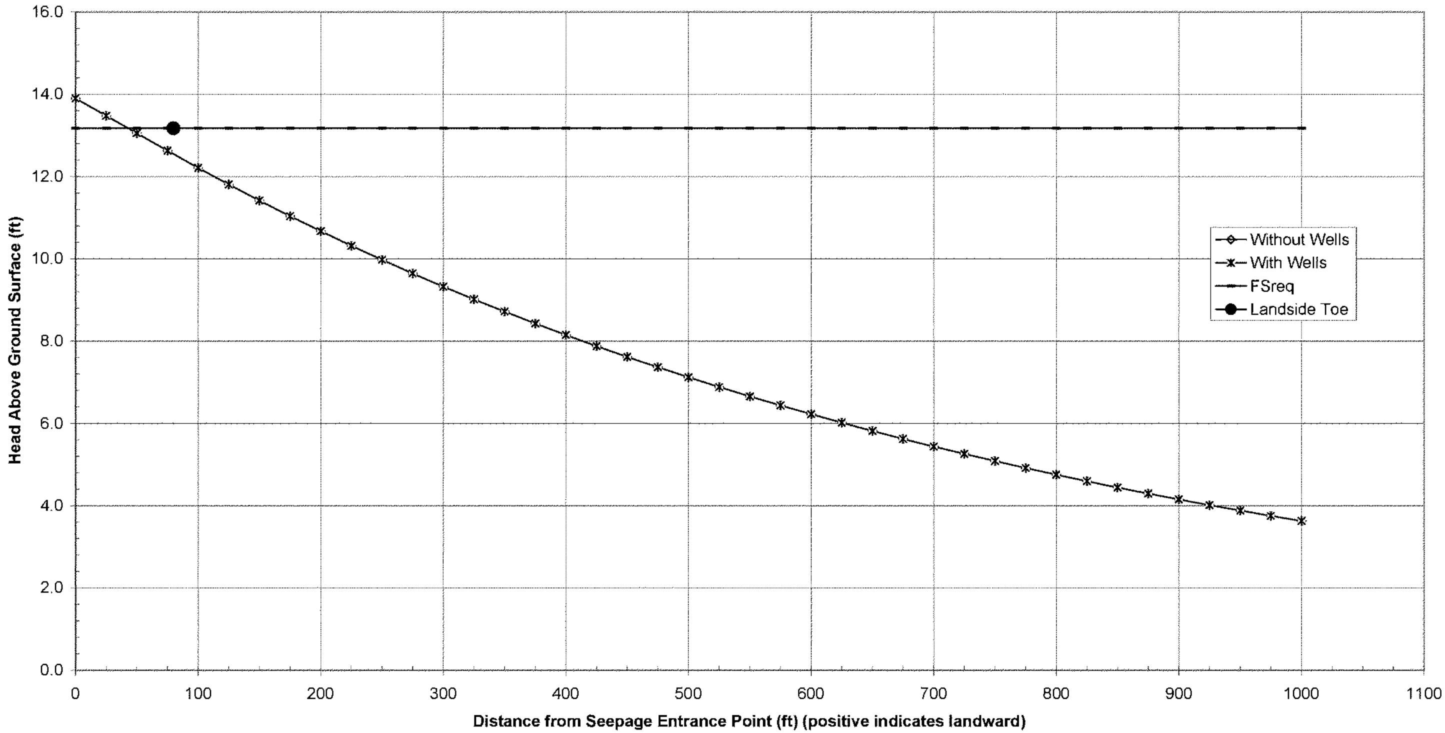
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

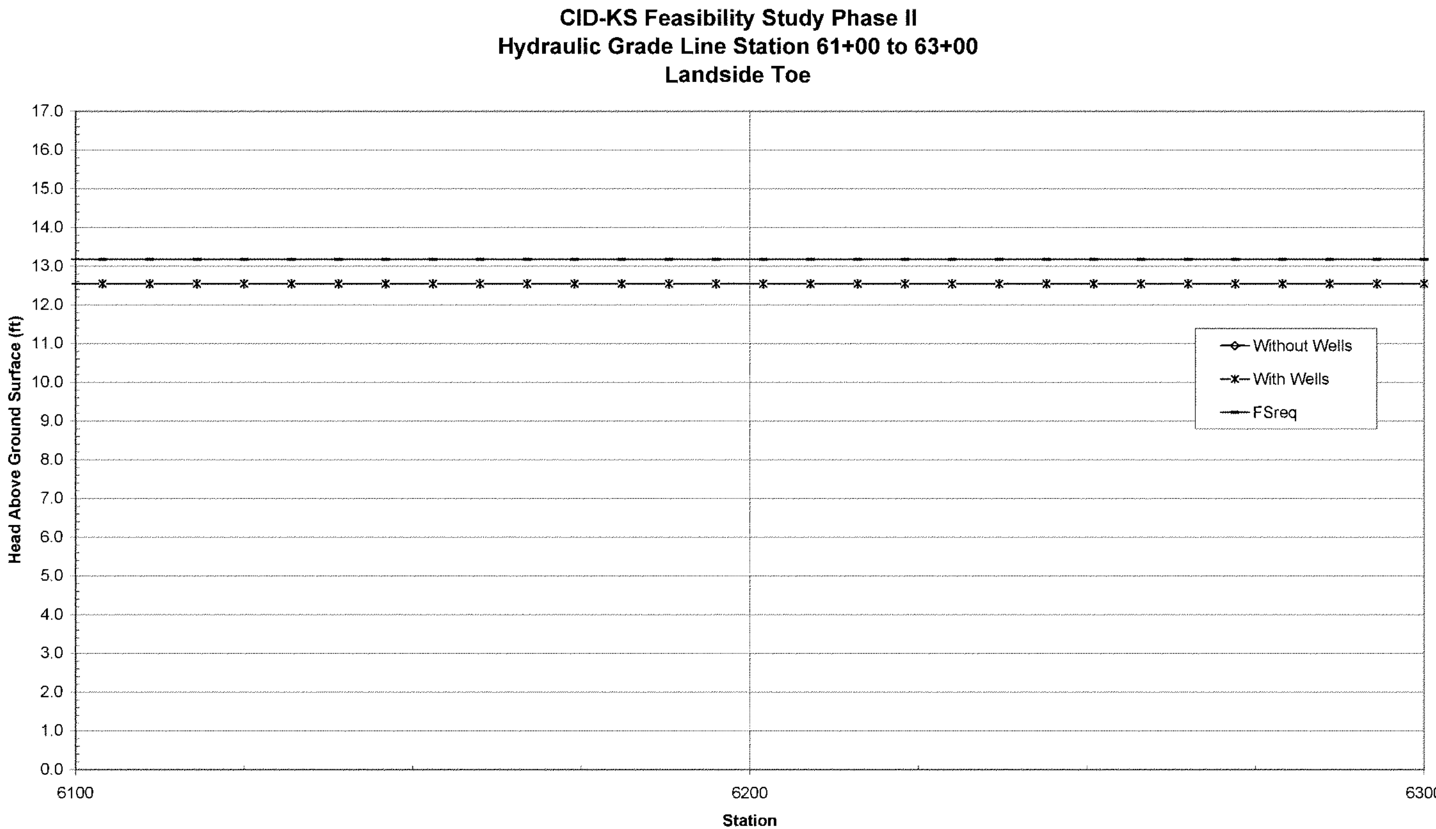
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	6100	13.9	0.0	13.9	13.17	1.00	0.56	1.52
2	25	6100	13.5	0.0	13.5	13.17	1.00	0.54	1.56
3	50	6100	13.1	0.0	13.1	13.17	1.00	0.52	1.61
4	75	6100	12.6	0.0	12.6	13.17	1.00	0.51	1.67
5	100	6100	12.2	0.0	12.2	13.17	1.00	0.49	1.73
6	125	6100	11.8	0.0	11.8	13.17	1.00	0.47	1.78
7	150	6100	11.4	0.0	11.4	13.17	1.00	0.46	1.85
8	175	6100	11.0	0.0	11.0	13.17	1.00	0.44	1.91
9	200	6100	10.7	0.0	10.7	13.17	1.00	0.43	1.97
10	225	6100	10.3	0.0	10.3	13.17	1.00	0.41	2.04
11	250	6100	10.0	0.0	10.0	13.17	1.00	0.40	2.11
12	275	6100	9.6	0.0	9.6	13.17	1.00	0.39	2.18
13	300	6100	9.3	0.0	9.3	13.17	1.00	0.37	2.26
14	325	6100	9.0	0.0	9.0	13.17	1.00	0.36	2.34
15	350	6100	8.7	0.0	8.7	13.17	1.00	0.35	2.42
16	375	6100	8.4	0.0	8.4	13.17	1.00	0.34	2.50
17	400	6100	8.1	0.0	8.1	13.17	1.00	0.33	2.59
18	425	6100	7.9	0.0	7.9	13.17	1.00	0.32	2.67
19	450	6100	7.6	0.0	7.6	13.17	1.00	0.30	2.77
20	475	6100	7.4	0.0	7.4	13.17	1.00	0.29	2.86
21	500	6100	7.1	0.0	7.1	13.17	1.00	0.28	2.96
22	525	6100	6.9	0.0	6.9	13.17	1.00	0.28	3.06
23	550	6100	6.7	0.0	6.7	13.17	1.00	0.27	3.17
24	575	6100	6.4	0.0	6.4	13.17	1.00	0.26	3.27
25	600	6100	6.2	0.0	6.2	13.17	1.00	0.25	3.39
26	625	6100	6.0	0.0	6.0	13.17	1.00	0.24	3.50
27	650	6100	5.8	0.0	5.8	13.17	1.00	0.23	3.62
28	675	6100	5.6	0.0	5.6	13.17	1.00	0.22	3.75
29	700	6100	5.4	0.0	5.4	13.17	1.00	0.22	3.88
30	725	6100	5.3	0.0	5.3	13.17	1.00	0.21	4.01
31	750	6100	5.1	0.0	5.1	13.17	1.00	0.20	4.15
32	775	6100	4.9	0.0	4.9	13.17	1.00	0.20	4.29
33	800	6100	4.8	0.0	4.8	13.17	1.00	0.19	4.43
34	825	6100	4.6	0.0	4.6	13.17	1.00	0.18	4.59
35	850	6100	4.4	0.0	4.4	13.17	1.00	0.18	4.74
36	875	6100	4.3	0.0	4.3	13.17	1.00	0.17	4.91
37	900	6100	4.2	0.0	4.2	13.17	1.00	0.17	5.07
38	925	6100	4.0	0.0	4.0	13.17	1.00	0.16	5.25
39	950	6100	3.9	0.0	3.9	13.17	1.00	0.16	5.43
40	975	6100	3.8	0.0	3.8	13.17	1.00	0.15	5.61
41	1000	6100	3.6	0.0	3.6	13.17	1.00	0.15	5.81

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	80	6090	12.5	0.0	12.5	13.17	1.00	0.50	1.68
2	80	6097	12.5	0.0	12.5	13.17	1.00	0.50	1.68
3	80	6104	12.5	0.0	12.5	13.17	1.00	0.50	1.68
4	80	6111	12.5	0.0	12.5	13.17	1.00	0.50	1.68
5	80	6118	12.5	0.0	12.5	13.17	1.00	0.50	1.68
6	80	6125	12.5	0.0	12.5	13.17	1.00	0.50	1.68
7	80	6132	12.5	0.0	12.5	13.17	1.00	0.50	1.68
8	80	6139	12.5	0.0	12.5	13.17	1.00	0.50	1.68
9	80	6146	12.5	0.0	12.5	13.17	1.00	0.50	1.68
10	80	6153	12.5	0.0	12.5	13.17	1.00	0.50	1.68
11	80	6160	12.5	0.0	12.5	13.17	1.00	0.50	1.68
12	80	6167	12.5	0.0	12.5	13.17	1.00	0.50	1.68
13	80	6174	12.5	0.0	12.5	13.17	1.00	0.50	1.68
14	80	6181	12.5	0.0	12.5	13.17	1.00	0.50	1.68
15	80	6188	12.5	0.0	12.5	13.17	1.00	0.50	1.68
16	80	6195	12.5	0.0	12.5	13.17	1.00	0.50	1.68
17	80	6202	12.5	0.0	12.5	13.17	1.00	0.50	1.68
18	80	6209	12.5	0.0	12.5	13.17	1.00	0.50	1.68
19	80	6216	12.5	0.0	12.5	13.17	1.00	0.50	1.68
20	80	6223	12.5	0.0	12.5	13.17	1.00	0.50	1.68
21	80	6230	12.5	0.0	12.5	13.17	1.00	0.50	1.68
22	80	6237	12.5	0.0	12.5	13.17	1.00	0.50	1.68
23	80	6244	12.5	0.0	12.5	13.17	1.00	0.50	1.68
24	80	6251	12.5	0.0	12.5	13.17	1.00	0.50	1.68
25	80	6258	12.5	0.0	12.5	13.17	1.00	0.50	1.68
26	80	6265	12.5	0.0	12.5	13.17	1.00	0.50	1.68
27	80	6272	12.5	0.0	12.5	13.17	1.00	0.50	1.68
28	80	6279	12.5	0.0	12.5	13.17	1.00	0.50	1.68
29	80	6286	12.5	0.0	12.5	13.17	1.00	0.50	1.68
30	80	6293	12.5	0.0	12.5	13.17	1.00	0.50	1.68
31	80	6300	12.5	0.0	12.5	13.17	1.00	0.50	1.68
32	80	6307	12.5	0.0	12.5	13.17	1.00	0.50	1.68
33	80	6314	12.5	0.0	12.5	13.17	1.00	0.50	1.68
34	80	6321	12.5	0.0	12.5	13.17	1.00	0.50	1.68
35	80	6328	12.5	0.0	12.5	13.17	1.00	0.50	1.68
36	80	6335	12.5	0.0	12.5	13.17	1.00	0.50	1.68
37	80	6342	12.5	0.0	12.5	13.17	1.00	0.50	1.68
38	80	6349	12.5	0.0	12.5	13.17	1.00	0.50	1.68
39	80	6356	12.5	0.0	12.5	13.17	1.00	0.50	1.68
40	80	6363	12.5	0.0	12.5	13.17	1.00	0.50	1.68
41	80	6370	12.5	0.0	12.5	13.17	1.00	0.50	1.68

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 61+00 to 63+00
Critical Station = 000+00




CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 61+00 to 63+00
Landside Toe
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
6100
6200
6300
Station
Without Wells
With Wells
FSreq

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	58	ft
h_0 =	14.57	ft
TOL =	764.5	ft elevation
Landside =	748.0	ft elevation
Bottom Blanket =	728	ft elevation
blanket =	20.0	ft
z, Landside Toe =	90	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	15.75	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
21	90	6250	748.0	1.09	0.35	1.01
22	90	6300	748.0	1.00	0.32	1.00
23	90	6350	748.0	0.96	0.31	1.00
24	90	6400	748.0	0.94	0.30	1.00
25	90	6450	748.0	0.93	0.29	1.00
26	90	6500	748.0	0.92	0.29	1.00
27	90	6550	748.0	0.92	0.29	1.00
28	90	6600	748.0	0.92	0.29	1.00
29	90	6650	748.0	0.93	0.29	1.00
30	90	6700	748.0	0.94	0.30	1.00
31	90	6750	748.0	0.96	0.31	1.00
32	90	6800	748.0	1.00	0.32	1.00
33	90	6850	748.0	1.09	0.35	1.01
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

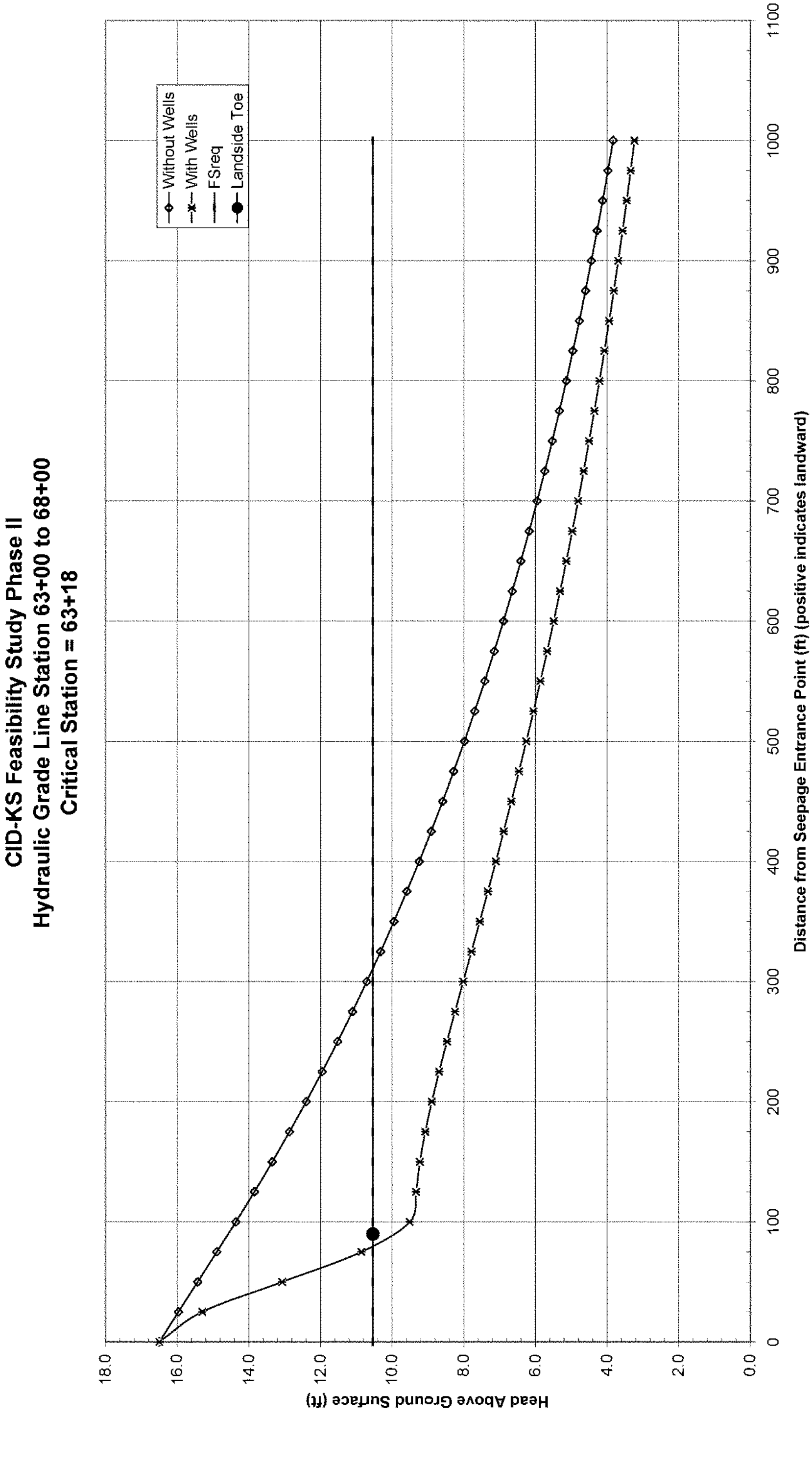
image well locations		
well	x'	y'
21	-90	6250
22	-90	6300
23	-90	6350
24	-90	6400
25	-90	6450
26	-90	6500
27	-90	6550
28	-90	6600
29	-90	6650
30	-90	6700
31	-90	6750
32	-90	6800
33	-90	6850
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

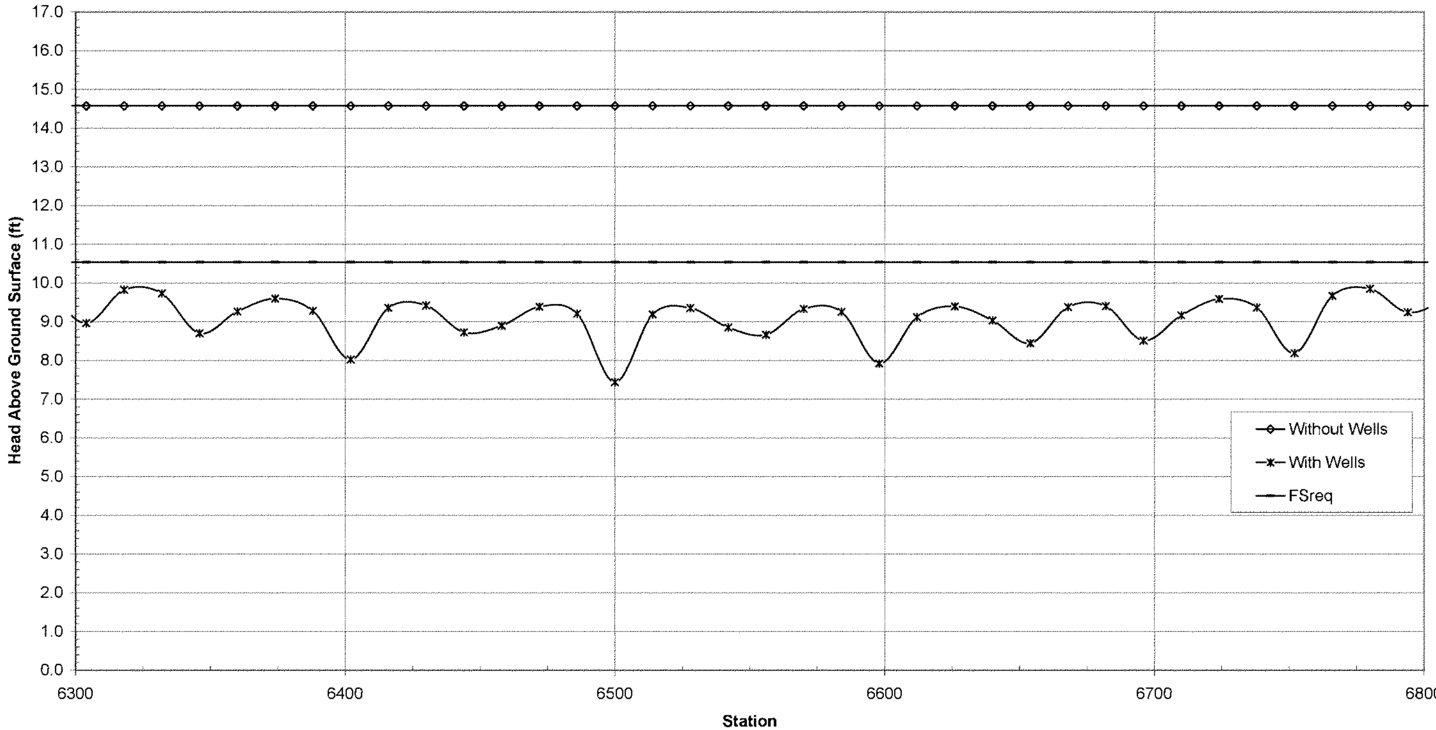
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	6318	16.5	0.0	16.5	10.54	1.00	0.83	1.02
2	25	6318	16.0	1.7	15.3	10.54	1.00	0.76	1.10
3	50	6318	15.4	3.4	13.1	10.54	1.00	0.65	1.29
4	75	6318	14.9	5.0	10.9	10.54	1.00	0.54	1.55
5	100	6318	14.4	5.8	9.5	10.54	1.00	0.48	1.77
6	125	6318	13.8	5.5	9.3	10.54	1.00	0.47	1.81
7	150	6318	13.3	5.1	9.2	10.54	1.00	0.46	1.83
8	175	6318	12.9	4.8	9.1	10.54	1.00	0.45	1.86
9	200	6318	12.4	4.5	8.9	10.54	1.00	0.44	1.90
10	225	6318	12.0	4.3	8.7	10.54	1.00	0.43	1.94
11	250	6318	11.5	4.1	8.5	10.54	1.00	0.42	1.99
12	275	6318	11.1	3.9	8.2	10.54	1.00	0.41	2.04
13	300	6318	10.7	3.7	8.0	10.54	1.00	0.40	2.10
14	325	6318	10.3	3.5	7.8	10.54	1.00	0.39	2.17
15	350	6318	9.9	3.4	7.6	10.54	1.00	0.38	2.23
16	375	6318	9.6	3.3	7.3	10.54	1.00	0.37	2.30
17	400	6318	9.2	3.1	7.1	10.54	1.00	0.36	2.37
18	425	6318	8.9	3.0	6.9	10.54	1.00	0.34	2.45
19	450	6318	8.6	2.9	6.7	10.54	1.00	0.33	2.53
20	475	6318	8.3	2.8	6.5	10.54	1.00	0.32	2.61
21	500	6318	8.0	2.7	6.3	10.54	1.00	0.31	2.69
22	525	6318	7.7	2.6	6.1	10.54	1.00	0.30	2.78
23	550	6318	7.4	2.6	5.9	10.54	1.00	0.29	2.88
24	575	6318	7.2	2.5	5.7	10.54	1.00	0.28	2.97
25	600	6318	6.9	2.4	5.5	10.54	1.00	0.27	3.07
26	625	6318	6.6	2.3	5.3	10.54	1.00	0.27	3.17
27	650	6318	6.4	2.3	5.1	10.54	1.00	0.26	3.28
28	675	6318	6.2	2.2	5.0	10.54	1.00	0.25	3.39
29	700	6318	6.0	2.1	4.8	10.54	1.00	0.24	3.50
30	725	6318	5.7	2.1	4.7	10.54	1.00	0.23	3.62
31	750	6318	5.5	2.0	4.5	10.54	1.00	0.23	3.75
32	775	6318	5.3	2.0	4.4	10.54	1.00	0.22	3.87
33	800	6318	5.1	1.9	4.2	10.54	1.00	0.21	4.00
34	825	6318	5.0	1.9	4.1	10.54	1.00	0.20	4.14
35	850	6318	4.8	1.8	3.9	10.54	1.00	0.20	4.28
36	875	6318	4.6	1.8	3.8	10.54	1.00	0.19	4.42
37	900	6318	4.4	1.7	3.7	10.54	1.00	0.18	4.57
38	925	6318	4.3	1.7	3.6	10.54	1.00	0.18	4.72
39	950	6318	4.1	1.7	3.5	10.54	1.00	0.17	4.88
40	975	6318	4.0	1.6	3.3	10.54	1.00	0.17	5.05
41	1000	6318	3.8	1.6	3.2	10.54	1.00	0.16	5.22

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	90	6290	14.6	5.8	9.8	10.54	1.00	0.49	1.72
2	90	6304	14.6	6.6	9.0	10.54	1.00	0.45	1.88
3	90	6318	14.6	5.7	9.8	10.54	1.00	0.49	1.72
4	90	6332	14.6	5.8	9.7	10.54	1.00	0.49	1.73
5	90	6346	14.6	6.9	8.7	10.54	1.00	0.44	1.94
6	90	6360	14.6	6.3	9.3	10.54	1.00	0.46	1.82
7	90	6374	14.6	6.0	9.6	10.54	1.00	0.48	1.76
8	90	6388	14.6	6.3	9.3	10.54	1.00	0.46	1.82
9	90	6402	14.6	7.5	8.0	10.54	1.00	0.40	2.10
10	90	6416	14.6	6.2	9.4	10.54	1.00	0.47	1.80
11	90	6430	14.6	6.2	9.4	10.54	1.00	0.47	1.79
12	90	6444	14.6	6.8	8.7	10.54	1.00	0.44	1.93
13	90	6458	14.6	6.7	8.9	10.54	1.00	0.44	1.89
14	90	6472	14.6	6.2	9.4	10.54	1.00	0.47	1.80
15	90	6486	14.6	6.4	9.2	10.54	1.00	0.46	1.83
16	90	6500	14.6	8.1	7.4	10.54	1.00	0.37	2.27
17	90	6514	14.6	6.4	9.2	10.54	1.00	0.46	1.83
18	90	6528	14.6	6.2	9.4	10.54	1.00	0.47	1.80
19	90	6542	14.6	6.7	8.9	10.54	1.00	0.44	1.90
20	90	6556	14.6	6.9	8.7	10.54	1.00	0.43	1.95
21	90	6570	14.6	6.2	9.3	10.54	1.00	0.47	1.81
22	90	6584	14.6	6.3	9.3	10.54	1.00	0.46	1.82
23	90	6598	14.6	7.7	7.9	10.54	1.00	0.40	2.13
24	90	6612	14.6	6.5	9.1	10.54	1.00	0.46	1.85
25	90	6626	14.6	6.2	9.4	10.54	1.00	0.47	1.79
26	90	6640	14.6	6.5	9.0	10.54	1.00	0.45	1.87
27	90	6654	14.6	7.1	8.4	10.54	1.00	0.42	2.00
28	90	6668	14.6	6.2	9.4	10.54	1.00	0.47	1.80
29	90	6682	14.6	6.2	9.4	10.54	1.00	0.47	1.79
30	90	6696	14.6	7.1	8.5	10.54	1.00	0.43	1.98
31	90	6710	14.6	6.4	9.2	10.54	1.00	0.46	1.84
32	90	6724	14.6	6.0	9.6	10.54	1.00	0.48	1.76
33	90	6738	14.6	6.2	9.4	10.54	1.00	0.47	1.80
34	90	6752	14.6	7.4	8.2	10.54	1.00	0.41	2.06
35	90	6766	14.6	5.9	9.7	10.54	1.00	0.48	1.74
36	90	6780	14.6	5.7	9.8	10.54	1.00	0.49	1.71
37	90	6794	14.6	6.3	9.2	10.54	1.00	0.46	1.82
38	90	6808	14.6	6.0	9.6	10.54	1.00	0.48	1.76
39	90	6822	14.6	5.2	10.3	10.54	1.00	0.52	1.63
40	90	6836	14.6	5.2	10.4	10.54	1.00	0.52	1.63
41	90	6850	14.6	7.0	8.6	10.54	1.00	0.43	1.96


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 63+00 to 68+00
Critical Station = 63+18
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 63+00 to 68+00
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	48	ft
h_b =	15.59	ft
TOL =	765	ft elevation
Landside =	747.0	ft elevation
Bottom Blanket =	718	ft elevation
blanket =	29.0	ft
z, Landside Toe =	100	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	7.60	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
30	90	6700	748.0	0.97	0.31	1.00
31	90	6750	748.0	0.91	0.29	1.00
32	90	6800	748.0	0.90	0.29	1.00
33	90	6850	748.0	0.95	0.30	1.00
34	100	7050	747.0	1.15	0.37	1.01
35	100	7250	747.0	1.20	0.38	1.01
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

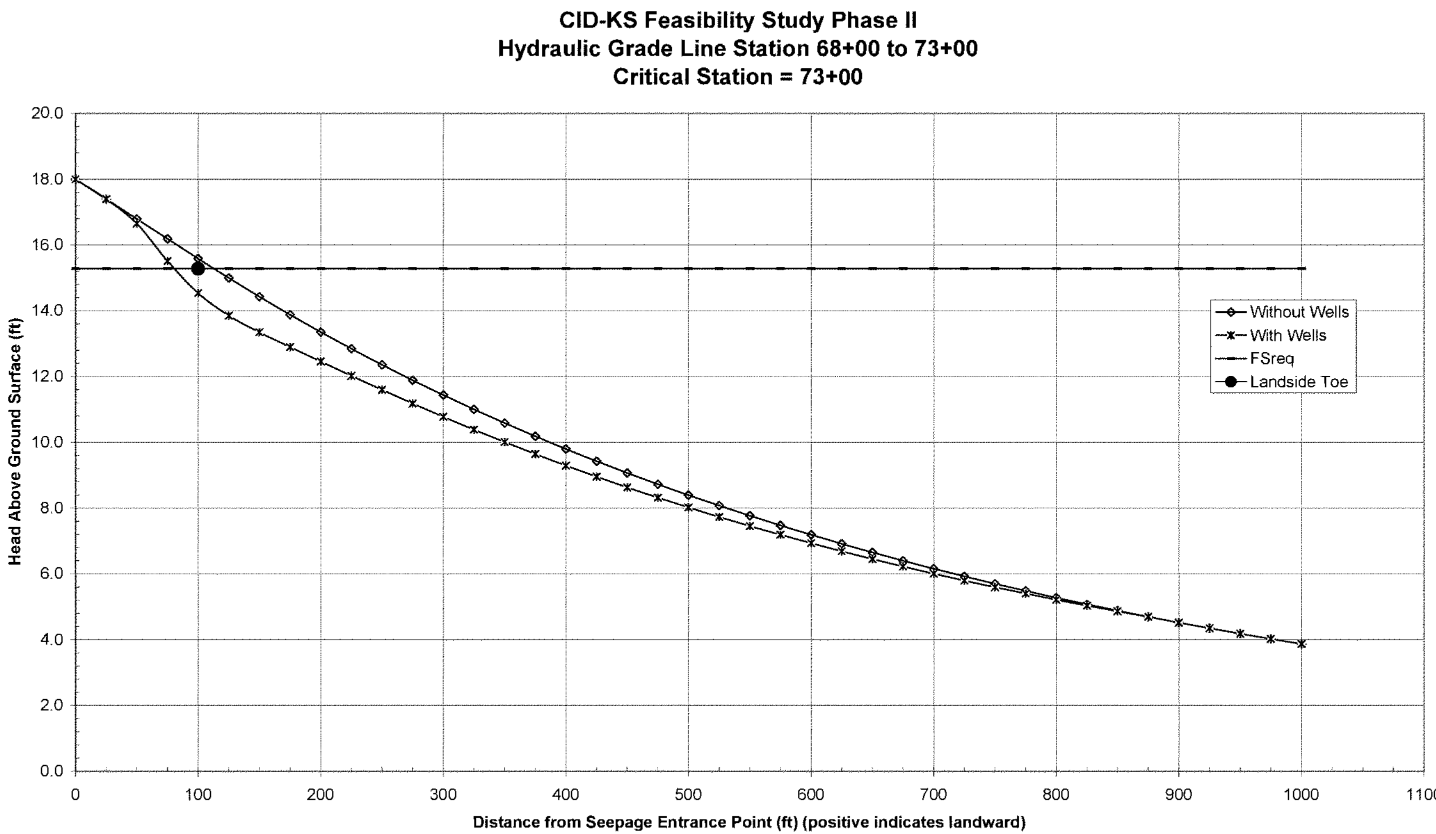
image well locations		
well	x'	y'
30	-90	6700
31	-90	6750
32	-90	6800
33	-90	6850
34	-100	7050
35	-100	7250
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

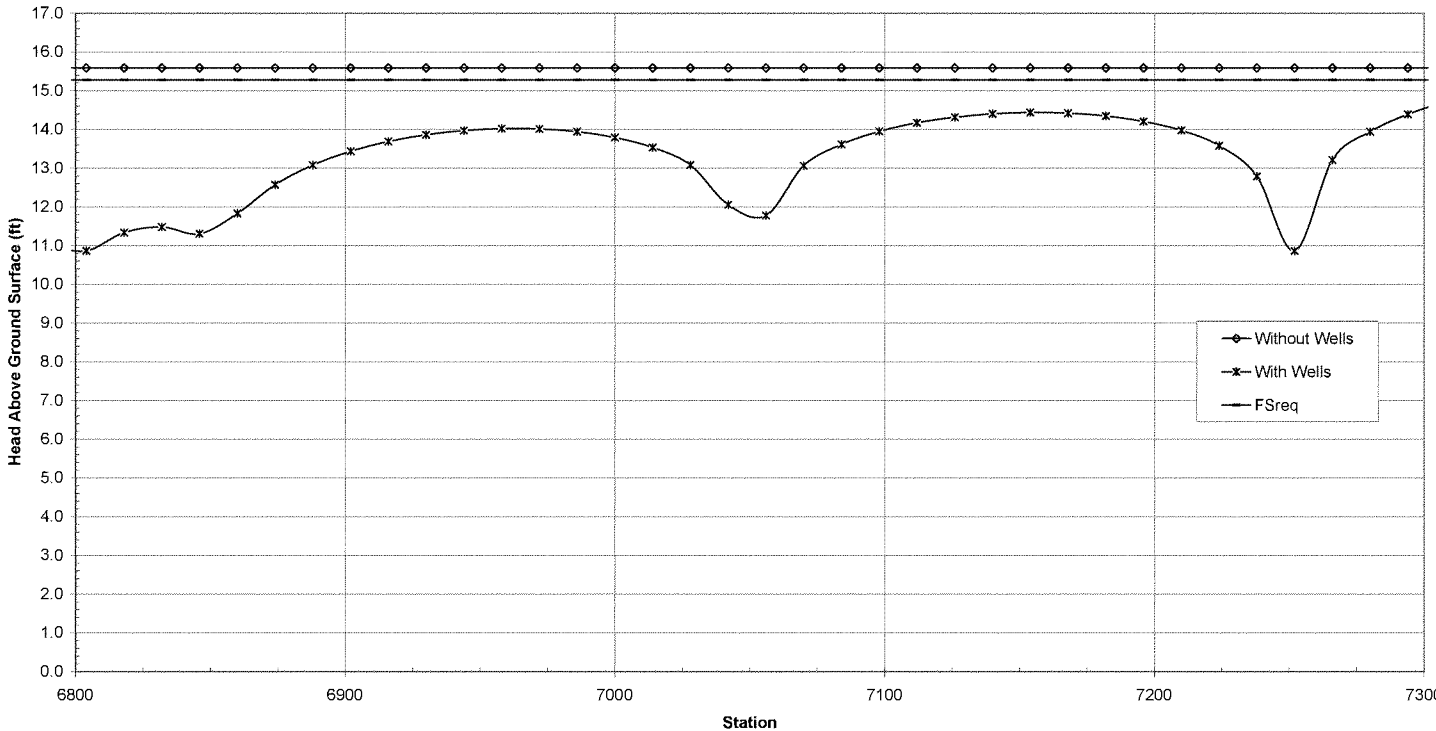
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	7300	18.0	0.0	18.0	15.28	1.00	0.62	1.36
2	25	7300	17.4	0.6	17.4	15.28	1.00	0.60	1.41
3	50	7300	16.8	1.1	16.6	15.28	1.00	0.57	1.47
4	75	7300	16.2	1.7	15.5	15.28	1.00	0.53	1.58
5	100	7300	15.6	2.0	14.5	15.28	1.00	0.50	1.68
6	125	7300	15.0	2.1	13.9	15.28	1.00	0.48	1.76
7	150	7300	14.4	2.1	13.3	15.28	1.00	0.46	1.83
8	175	7300	13.9	2.0	12.9	15.28	1.00	0.44	1.90
9	200	7300	13.4	1.9	12.5	15.28	1.00	0.43	1.96
10	225	7300	12.8	1.8	12.0	15.28	1.00	0.41	2.03
11	250	7300	12.4	1.8	11.6	15.28	1.00	0.40	2.11
12	275	7300	11.9	1.7	11.2	15.28	1.00	0.39	2.19
13	300	7300	11.4	1.7	10.8	15.28	1.00	0.37	2.27
14	325	7300	11.0	1.6	10.4	15.28	1.00	0.36	2.35
15	350	7300	10.6	1.6	10.0	15.28	1.00	0.35	2.44
16	375	7300	10.2	1.5	9.6	15.28	1.00	0.33	2.53
17	400	7300	9.8	1.5	9.3	15.28	1.00	0.32	2.63
18	425	7300	9.4	1.5	9.0	15.28	1.00	0.31	2.73
19	450	7300	9.1	1.4	8.6	15.28	1.00	0.30	2.83
20	475	7300	8.7	1.4	8.3	15.28	1.00	0.29	2.94
21	500	7300	8.4	1.4	8.0	15.28	1.00	0.28	3.05
22	525	7300	8.1	1.3	7.7	15.28	1.00	0.27	3.16
23	550	7300	7.8	1.3	7.5	15.28	1.00	0.26	3.28
24	575	7300	7.5	1.3	7.2	15.28	1.00	0.25	3.40
25	600	7300	7.2	1.3	6.9	15.28	1.00	0.24	3.53
26	625	7300	6.9	1.2	6.7	15.28	1.00	0.23	3.65
27	650	7300	6.7	1.2	6.5	15.28	1.00	0.22	3.79
28	675	7300	6.4	1.2	6.2	15.28	1.00	0.21	3.93
29	700	7300	6.2	1.2	6.0	15.28	1.00	0.21	4.07
30	725	7300	5.9	1.1	5.8	15.28	1.00	0.20	4.22
31	750	7300	5.7	1.1	5.6	15.28	1.00	0.19	4.37
32	775	7300	5.5	1.1	5.4	15.28	1.00	0.19	4.52
33	800	7300	5.3	1.1	5.2	15.28	1.00	0.18	4.69
34	825	7300	5.1	1.0	5.0	15.28	1.00	0.17	4.85
35	850	7300	4.9	1.0	4.9	15.28	1.00	0.17	5.02
36	875	7300	4.7	1.0	4.7	15.28	1.00	0.16	5.20
37	900	7300	4.5	1.0	4.5	15.28	1.00	0.16	5.41
38	925	7300	4.3	1.0	4.3	15.28	1.00	0.15	5.62
39	950	7300	4.2	0.9	4.2	15.28	1.00	0.14	5.84
40	975	7300	4.0	0.9	4.0	15.28	1.00	0.14	6.07
41	1000	7300	3.9	0.9	3.9	15.28	1.00	0.13	6.31

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	100	6790	15.6	5.6	11.0	15.28	1.00	0.38	2.22
2	100	6804	15.6	5.7	10.9	15.28	1.00	0.37	2.25
3	100	6818	15.6	5.3	11.3	15.28	1.00	0.39	2.16
4	100	6832	15.6	5.1	11.5	15.28	1.00	0.40	2.13
5	100	6846	15.6	5.3	11.3	15.28	1.00	0.39	2.16
6	100	6860	15.6	4.8	11.8	15.28	1.00	0.41	2.07
7	100	6874	15.6	4.0	12.6	15.28	1.00	0.43	1.94
8	100	6888	15.6	3.5	13.1	15.28	1.00	0.45	1.87
9	100	6902	15.6	3.2	13.4	15.28	1.00	0.46	1.82
10	100	6916	15.6	2.9	13.7	15.28	1.00	0.47	1.79
11	100	6930	15.6	2.7	13.9	15.28	1.00	0.48	1.76
12	100	6944	15.6	2.6	14.0	15.28	1.00	0.48	1.75
13	100	6958	15.6	2.6	14.0	15.28	1.00	0.48	1.74
14	100	6972	15.6	2.6	14.0	15.28	1.00	0.48	1.74
15	100	6986	15.6	2.6	13.9	15.28	1.00	0.48	1.75
16	100	7000	15.6	2.8	13.8	15.28	1.00	0.48	1.77
17	100	7014	15.6	3.1	13.5	15.28	1.00	0.47	1.81
18	100	7028	15.6	3.5	13.1	15.28	1.00	0.45	1.87
19	100	7042	15.6	4.5	12.1	15.28	1.00	0.42	2.03
20	100	7056	15.6	4.8	11.8	15.28	1.00	0.41	2.08
21	100	7070	15.6	3.5	13.1	15.28	1.00	0.45	1.87
22	100	7084	15.6	3.0	13.6	15.28	1.00	0.47	1.80
23	100	7098	15.6	2.6	13.9	15.28	1.00	0.48	1.75
24	100	7112	15.6	2.4	14.2	15.28	1.00	0.49	1.73
25	100	7126	15.6	2.3	14.3	15.28	1.00	0.49	1.71
26	100	7140	15.6	2.2	14.4	15.28	1.00	0.50	1.70
27	100	7154	15.6	2.2	14.4	15.28	1.00	0.50	1.69
28	100	7168	15.6	2.2	14.4	15.28	1.00	0.50	1.70
29	100	7182	15.6	2.2	14.3	15.28	1.00	0.49	1.70
30	100	7196	15.6	2.4	14.2	15.28	1.00	0.49	1.72
31	100	7210	15.6	2.6	14.0	15.28	1.00	0.48	1.75
32	100	7224	15.6	3.0	13.6	15.28	1.00	0.47	1.80
33	100	7238	15.6	3.8	12.8	15.28	1.00	0.44	1.91
34	100	7252	15.6	5.7	10.9	15.28	1.00	0.37	2.25
35	100	7266	15.6	3.4	13.2	15.28	1.00	0.46	1.85
36	100	7280	15.6	2.6	13.9	15.28	1.00	0.48	1.75
37	100	7294	15.6	2.2	14.4	15.28	1.00	0.50	1.70
38	100	7308	15.6	1.9	14.7	15.28	1.00	0.51	1.66
39	100	7322	15.6	1.6	15.0	15.28	1.00	0.52	1.63
40	100	7336	15.6	1.4	15.2	15.28	1.00	0.52	1.61
41	100	7350	15.6	1.3	15.3	15.28	1.00	0.53	1.60


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 68+00 to 73+00
Critical Station = 73+00
Head Above Ground Surface (ft)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100
Distance from Seepage Entrance Point (ft) (positive indicates landward)
Without Wells
With Wells
FSreq
Landside Toe

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 68+00 to 73+00
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	53	ft
h_0 =	17.02	ft
TOL =	765.2	ft elevation
Landside =	746.0	ft elevation
Bottom Blanket =	723	ft elevation
blanket =	23.0	ft
z, Landside Toe =	100	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{eq} =	1.6	
Efficiency =	0.8	
Total Flow =	15.96	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	V_w (ft/s)	H_w (ft)
8	152	7930	739.0	1.71	0.54	1.01
9	153	8050	740.0	1.68	0.53	1.01
34	100	7050	747.0	1.21	0.39	1.01
35	100	7250	747.0	0.81	0.26	1.00
36	100	7275	746.0	0.79	0.25	1.00
37	100	7300	746.0	0.75	0.24	1.00
38	100	7325	746.0	0.73	0.23	1.00
39	100	7350	746.0	0.71	0.23	1.00
40	100	7375	746.0	0.71	0.23	1.00
41	100	7400	746.0	0.71	0.23	1.00
42	100	7425	746.0	0.72	0.23	1.00
43	100	7450	746.0	0.73	0.23	1.00
44	100	7470	746.0	0.74	0.24	1.00
45	100	7480	746.0	0.78	0.25	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

image well locations		
well	x'	y'
8	-152	7930
9	-153	8050
34	-100	7050
35	-100	7250
36	-100	7275
37	-100	7300
38	-100	7325
39	-100	7350
40	-100	7375
41	-100	7400
42	-100	7425
43	-100	7450
44	-100	7470
45	-100	7480
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

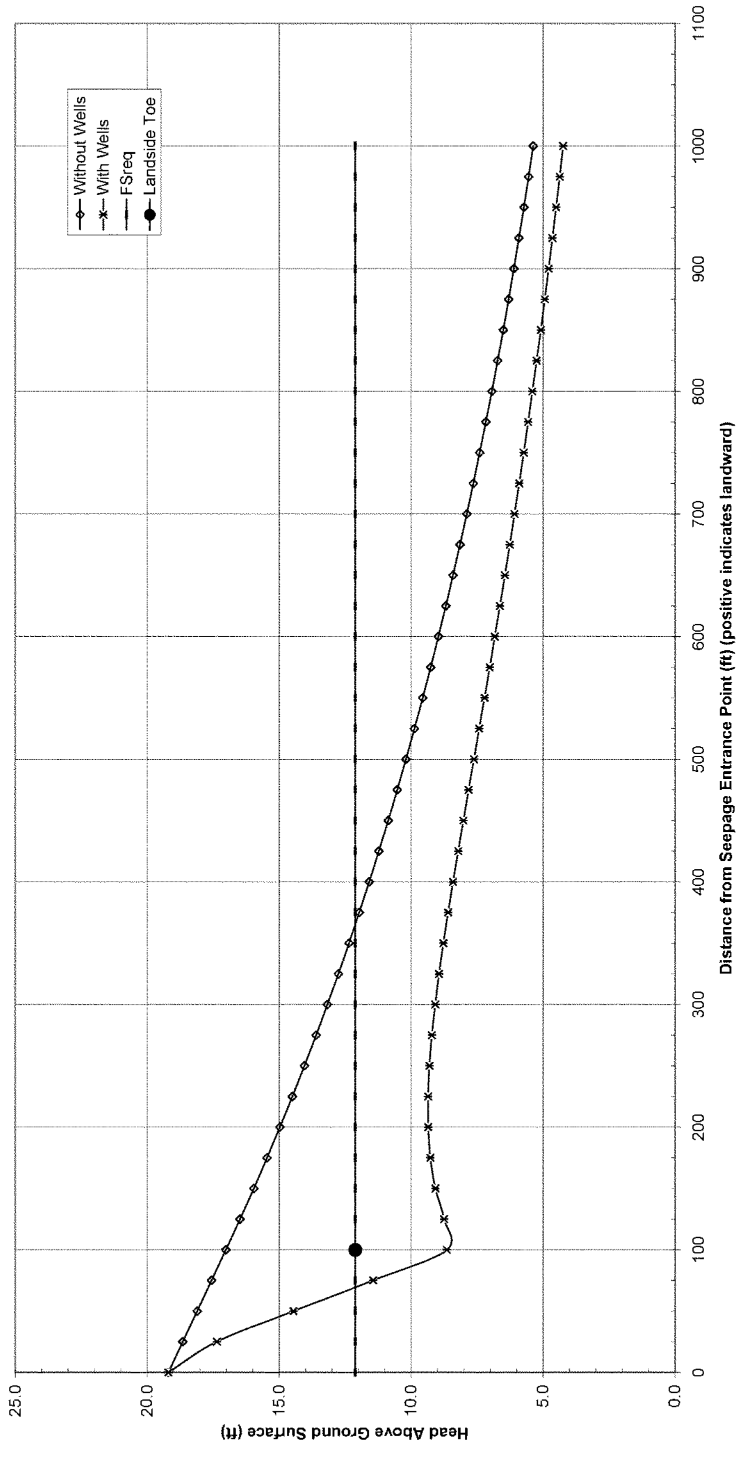
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

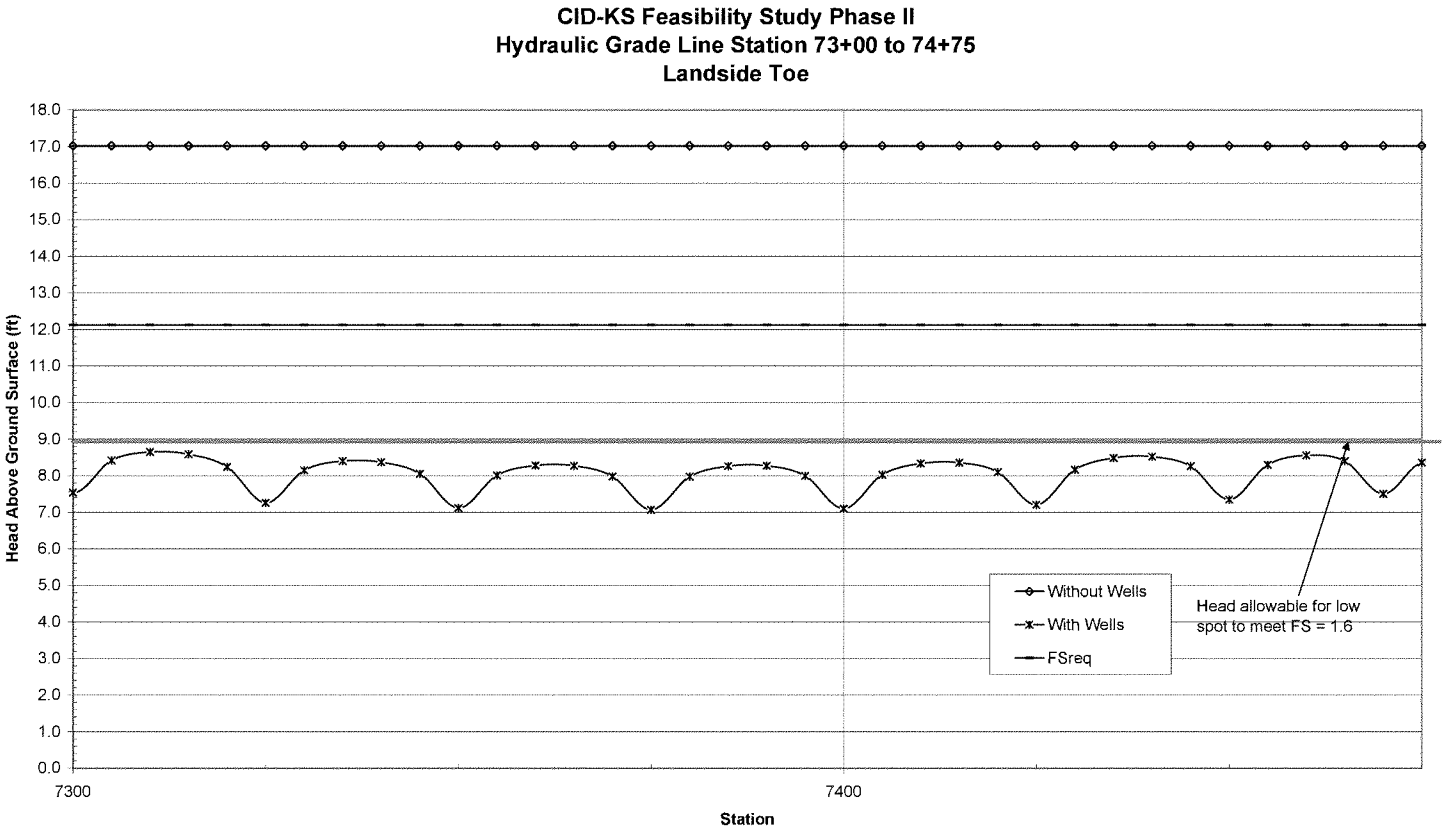
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	7310	19.2	0.0	19.2	12.12	1.00	0.83	1.01
2	25	7310	18.7	2.3	17.4	12.12	1.00	0.75	1.12
3	50	7310	18.1	4.7	14.5	12.12	1.00	0.63	1.34
4	75	7310	17.6	7.1	11.4	12.12	1.00	0.50	1.69
5	100	7310	17.0	9.4	8.6	12.12	1.00	0.38	2.24
6	125	7310	16.5	8.7	8.8	12.12	1.00	0.38	2.21
7	150	7310	16.0	7.9	9.1	12.12	1.00	0.39	2.13
8	175	7310	15.5	7.2	9.3	12.12	1.00	0.40	2.09
9	200	7310	15.0	6.6	9.4	12.12	1.00	0.41	2.07
10	225	7310	14.5	6.1	9.4	12.12	1.00	0.41	2.07
11	250	7310	14.0	5.7	9.3	12.12	1.00	0.40	2.08
12	275	7310	13.6	5.4	9.2	12.12	1.00	0.40	2.10
13	300	7310	13.2	5.1	9.1	12.12	1.00	0.40	2.13
14	325	7310	12.8	4.8	8.9	12.12	1.00	0.39	2.17
15	350	7310	12.4	4.6	8.8	12.12	1.00	0.38	2.21
16	375	7310	12.0	4.4	8.6	12.12	1.00	0.37	2.25
17	400	7310	11.6	4.2	8.4	12.12	1.00	0.37	2.30
18	425	7310	11.2	4.0	8.2	12.12	1.00	0.36	2.36
19	450	7310	10.9	3.8	8.0	12.12	1.00	0.35	2.42
20	475	7310	10.5	3.7	7.8	12.12	1.00	0.34	2.48
21	500	7310	10.2	3.6	7.6	12.12	1.00	0.33	2.54
22	525	7310	9.9	3.5	7.4	12.12	1.00	0.32	2.61
23	550	7310	9.6	3.3	7.2	12.12	1.00	0.31	2.68
24	575	7310	9.3	3.2	7.0	12.12	1.00	0.31	2.76
25	600	7310	9.0	3.1	6.8	12.12	1.00	0.30	2.84
26	625	7310	8.7	3.0	6.6	12.12	1.00	0.29	2.92
27	650	7310	8.4	3.0	6.5	12.12	1.00	0.28	3.01
28	675	7310	8.1	2.9	6.3	12.12	1.00	0.27	3.09
29	700	7310	7.9	2.8	6.1	12.12	1.00	0.26	3.19
30	725	7310	7.6	2.7	5.9	12.12	1.00	0.26	3.28
31	750	7310	7.4	2.7	5.7	12.12	1.00	0.25	3.38
32	775	7310	7.2	2.6	5.6	12.12	1.00	0.24	3.48
33	800	7310	6.9	2.5	5.4	12.12	1.00	0.23	3.59
34	825	7310	6.7	2.5	5.2	12.12	1.00	0.23	3.70
35	850	7310	6.5	2.4	5.1	12.12	1.00	0.22	3.81
36	875	7310	6.3	2.4	4.9	12.12	1.00	0.21	3.93
37	900	7310	6.1	2.3	4.8	12.12	1.00	0.21	4.05
38	925	7310	6.9	2.3	4.6	12.12	1.00	0.20	4.17
39	950	7310	5.7	2.2	4.5	12.12	1.00	0.20	4.30
40	975	7310	5.5	2.2	4.4	12.12	1.00	0.19	4.43
41	1000	7310	5.4	2.1	4.2	12.12	1.00	0.18	4.57

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	100	7295	17.0	9.4	8.6	12.12	1.00	0.37	2.26
2	100	7300	17.0	10.5	7.5	12.12	1.00	0.33	2.57
3	100	7305	17.0	9.6	8.4	12.12	1.00	0.37	2.30
4	100	7310	17.0	9.4	8.6	12.12	1.00	0.38	2.24
5	100	7315	17.0	9.4	8.6	12.12	1.00	0.37	2.26
6	100	7320	17.0	9.8	8.2	12.12	1.00	0.36	2.35
7	100	7325	17.0	10.8	7.3	12.12	1.00	0.32	2.67
8	100	7330	17.0	9.9	8.1	12.12	1.00	0.35	2.38
9	100	7335	17.0	9.6	8.4	12.12	1.00	0.37	2.31
10	100	7340	17.0	9.7	8.4	12.12	1.00	0.36	2.32
11	100	7345	17.0	10.0	8.1	12.12	1.00	0.35	2.41
12	100	7350	17.0	10.9	7.1	12.12	1.00	0.31	2.73
13	100	7355	17.0	10.0	8.0	12.12	1.00	0.35	2.42
14	100	7360	17.0	9.7	8.3	12.12	1.00	0.36	2.34
15	100	7365	17.0	9.8	8.3	12.12	1.00	0.36	2.34
16	100	7370	17.0	10.0	8.0	12.12	1.00	0.35	2.43
17	100	7375	17.0	11.0	7.1	12.12	1.00	0.31	2.75
18	100	7380	17.0	10.0	8.0	12.12	1.00	0.35	2.43
19	100	7385	17.0	9.8	8.3	12.12	1.00	0.36	2.35
20	100	7390	17.0	9.8	8.3	12.12	1.00	0.36	2.34
21	100	7395	17.0	10.0	8.0	12.12	1.00	0.35	2.43
22	100	7400	17.0	10.9	7.1	12.12	1.00	0.31	2.73
23	100	7405	17.0	10.0	8.0	12.12	1.00	0.35	2.42
24	100	7410	17.0	9.7	8.3	12.12	1.00	0.36	2.33
25	100	7415	17.0	9.7	8.4	12.12	1.00	0.36	2.32
26	100	7420	17.0	9.9	8.1	12.12	1.00	0.35	2.39
27	100	7425	17.0	10.8	7.2	12.12	1.00	0.31	2.69
28	100	7430	17.0	9.9	8.2	12.12	1.00	0.35	2.38
29	100	7435	17.0	9.5	8.5	12.12	1.00	0.37	2.28
30	100	7440	17.0	9.5	8.5	12.12	1.00	0.37	2.28
31	100	7445	17.0	9.8	8.3	12.12	1.00	0.36	2.35
32	100	7450	17.0	10.7	7.3	12.12	1.00	0.32	2.64
33	100	7455	17.0	9.7	8.3	12.12	1.00	0.36	2.34
34	100	7460	17.0	9.5	8.6	12.12	1.00	0.37	2.27
35	100	7465	17.0	9.6	8.4	12.12	1.00	0.37	2.31
36	100	7470	17.0	10.5	7.5	12.12	1.00	0.33	2.58
37	100	7475	17.0	9.7	8.4	12.12	1.00	0.36	2.32
38	100	7480	17.0	10.0	8.0	12.12	1.00	0.35	2.42
39	100	7485	17.0	8.5	9.5	12.12	1.00	0.41	2.03
40	100	7490	17.0	7.6	10.4	12.12	1.00	0.45	1.87
41	100	7495	17.0	7.0	11.0	12.12	1.00	0.48	1.77

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 73+00 to 74+75
Critical Station = 73+10




CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 73+00 to 74+75
Landside Toe
Head Above Ground Surface (ft)
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
7300
7400
Station
Without Wells
With Wells
FSreq
Head allowable for low spot to meet FS = 1.6

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	52	ft
h_o =	4.78	ft
TOL =	765.5	ft elevation
Landside =	760.0	ft elevation
Bottom Blanket =	722	ft elevation
blanket =	38.0	ft
z, Landside Toe =	150	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.64	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	0.00	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00

1.00 **<---Input H_w AVG after any changes are made to well parameters**

image well locations		
well	x'	y'
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

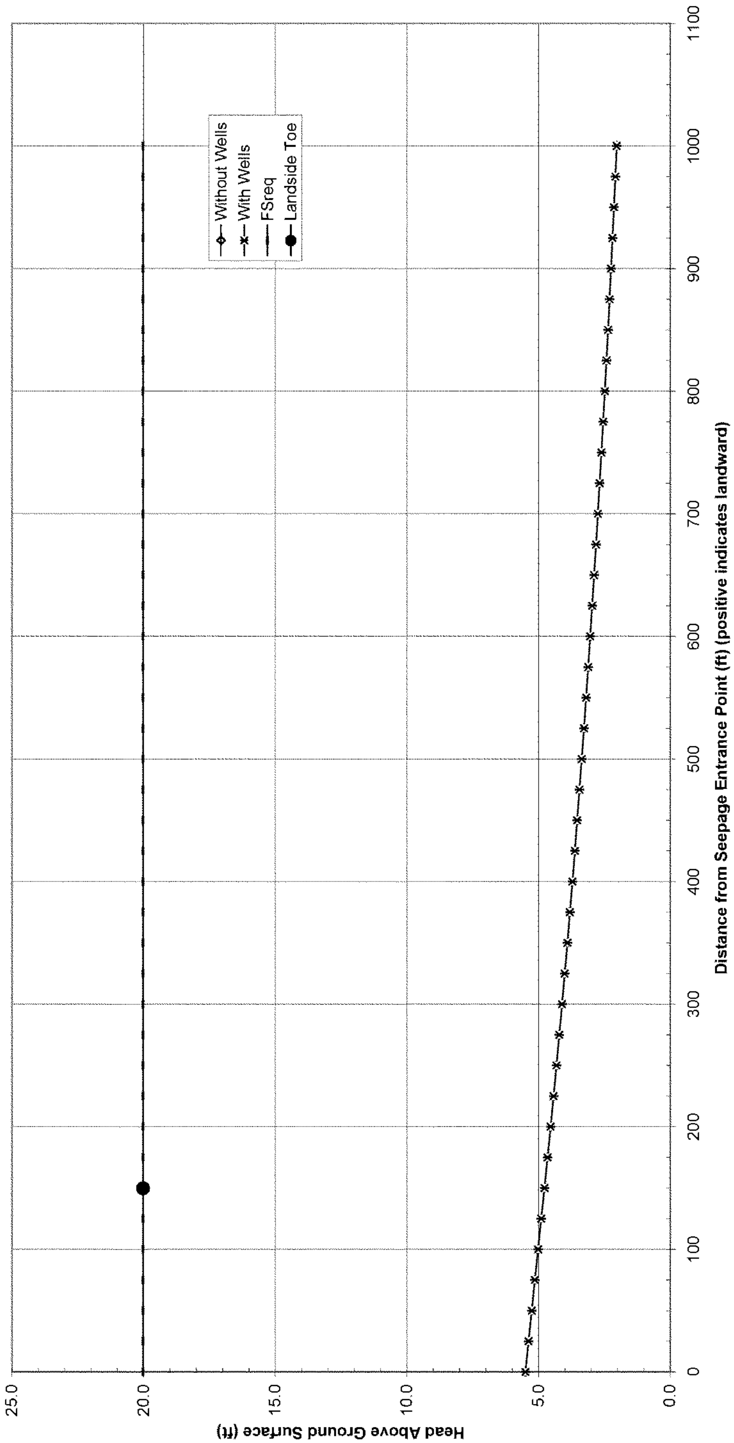
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	7475	5.5	0.0	5.5	20.02	1.00	0.14	5.82
2	25	7475	5.4	0.0	5.4	20.02	1.00	0.14	5.95
3	50	7475	5.3	0.0	5.3	20.02	1.00	0.14	6.09
4	75	7475	5.1	0.0	5.1	20.02	1.00	0.14	6.23
5	100	7475	5.0	0.0	5.0	20.02	1.00	0.13	6.38
6	125	7475	4.9	0.0	4.9	20.02	1.00	0.13	6.54
7	150	7475	4.8	0.0	4.8	20.02	1.00	0.13	6.70
8	175	7475	4.7	0.0	4.7	20.02	1.00	0.12	6.87
9	200	7475	4.5	0.0	4.5	20.02	1.00	0.12	7.05
10	225	7475	4.4	0.0	4.4	20.02	1.00	0.12	7.23
11	250	7475	4.3	0.0	4.3	20.02	1.00	0.11	7.41
12	275	7475	4.2	0.0	4.2	20.02	1.00	0.11	7.60
13	300	7475	4.1	0.0	4.1	20.02	1.00	0.11	7.79
14	325	7475	4.0	0.0	4.0	20.02	1.00	0.11	7.99
15	350	7475	3.9	0.0	3.9	20.02	1.00	0.10	8.20
16	375	7475	3.8	0.0	3.8	20.02	1.00	0.10	8.41
17	400	7475	3.7	0.0	3.7	20.02	1.00	0.10	8.62
18	425	7475	3.6	0.0	3.6	20.02	1.00	0.10	8.84
19	450	7475	3.5	0.0	3.5	20.02	1.00	0.09	9.06
20	475	7475	3.4	0.0	3.4	20.02	1.00	0.09	9.30
21	500	7475	3.4	0.0	3.4	20.02	1.00	0.09	9.53
22	525	7475	3.3	0.0	3.3	20.02	1.00	0.09	9.77
23	550	7475	3.2	0.0	3.2	20.02	1.00	0.08	10.02
24	575	7475	3.1	0.0	3.1	20.02	1.00	0.08	10.28
25	600	7475	3.0	0.0	3.0	20.02	1.00	0.08	10.54
26	625	7475	3.0	0.0	3.0	20.02	1.00	0.08	10.81
27	650	7475	2.9	0.0	2.9	20.02	1.00	0.08	11.08
28	675	7475	2.8	0.0	2.8	20.02	1.00	0.07	11.37
29	700	7475	2.7	0.0	2.7	20.02	1.00	0.07	11.66
30	725	7475	2.7	0.0	2.7	20.02	1.00	0.07	11.96
31	750	7475	2.6	0.0	2.6	20.02	1.00	0.07	12.26
32	775	7475	2.5	0.0	2.5	20.02	1.00	0.07	12.57
33	800	7475	2.5	0.0	2.5	20.02	1.00	0.07	12.89
34	825	7475	2.4	0.0	2.4	20.02	1.00	0.06	13.22
35	850	7475	2.4	0.0	2.4	20.02	1.00	0.06	13.56
36	875	7475	2.3	0.0	2.3	20.02	1.00	0.06	13.90
37	900	7475	2.2	0.0	2.2	20.02	1.00	0.06	14.25
38	925	7475	2.2	0.0	2.2	20.02	1.00	0.06	14.62
39	950	7475	2.1	0.0	2.1	20.02	1.00	0.06	14.99
40	975	7475	2.1	0.0	2.1	20.02	1.00	0.05	15.37
41	1000	7475	2.0	0.0	2.0	20.02	1.00	0.05	15.76

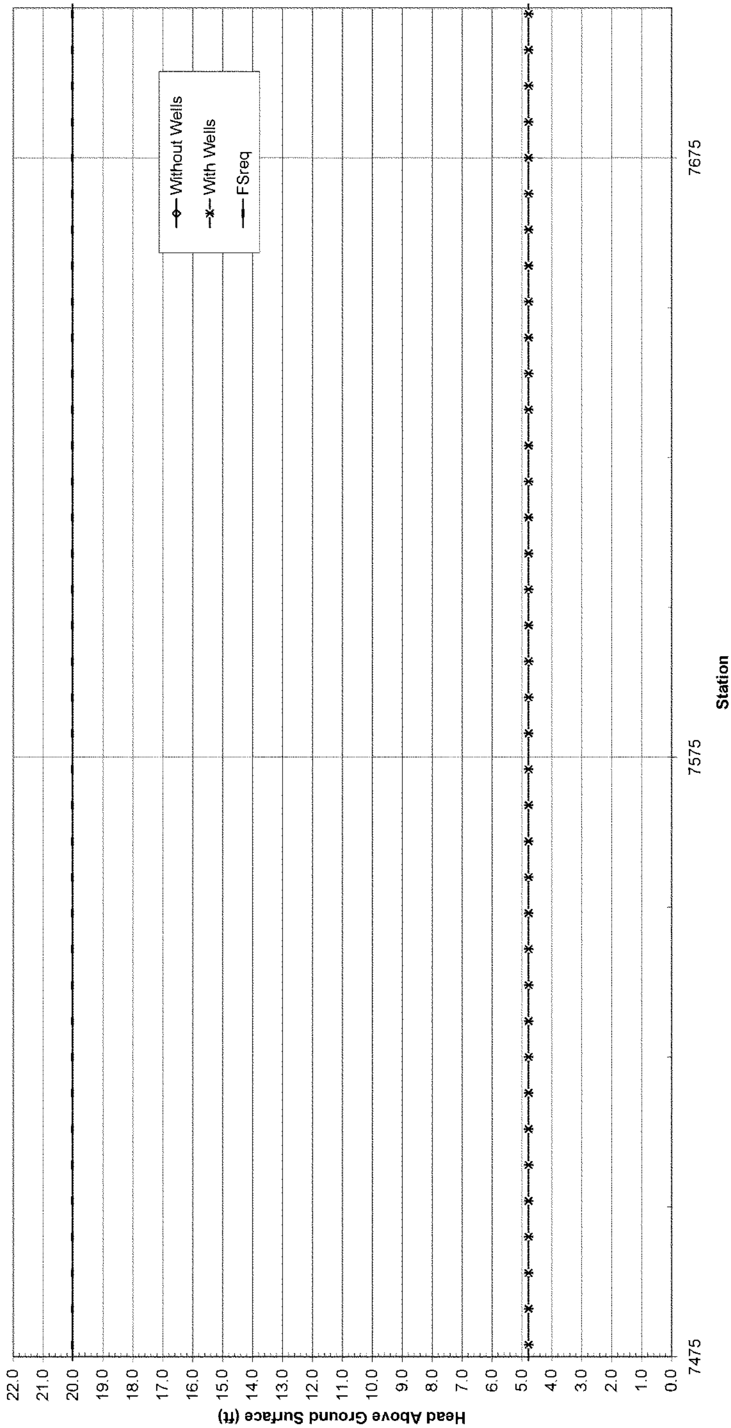
Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	150	7465	4.8	0.0	4.8	20.02	1.00	0.13	6.70
2	150	7471	4.8	0.0	4.8	20.02	1.00	0.13	6.70
3	150	7477	4.8	0.0	4.8	20.02	1.00	0.13	6.70
4	150	7483	4.8	0.0	4.8	20.02	1.00	0.13	6.70
5	150	7489	4.8	0.0	4.8	20.02	1.00	0.13	6.70
6	150	7495	4.8	0.0	4.8	20.02	1.00	0.13	6.70
7	150	7501	4.8	0.0	4.8	20.02	1.00	0.13	6.70
8	150	7507	4.8	0.0	4.8	20.02	1.00	0.13	6.70
9	150	7513	4.8	0.0	4.8	20.02	1.00	0.13	6.70
10	150	7519	4.8	0.0	4.8	20.02	1.00	0.13	6.70
11	150	7525	4.8	0.0	4.8	20.02	1.00	0.13	6.70
12	150	7531	4.8	0.0	4.8	20.02	1.00	0.13	6.70
13	150	7537	4.8	0.0	4.8	20.02	1.00	0.13	6.70
14	150	7543	4.8	0.0	4.8	20.02	1.00	0.13	6.70
15	150	7549	4.8	0.0	4.8	20.02	1.00	0.13	6.70
16	150	7555	4.8	0.0	4.8	20.02	1.00	0.13	6.70
17	150	7561	4.8	0.0	4.8	20.02	1.00	0.13	6.70
18	150	7567	4.8	0.0	4.8	20.02	1.00	0.13	6.70
19	150	7573	4.8	0.0	4.8	20.02	1.00	0.13	6.70
20	150	7579	4.8	0.0	4.8	20.02	1.00	0.13	6.70
21	150	7585	4.8	0.0	4.8	20.02	1.00	0.13	6.70
22	150	7591	4.8	0.0	4.8	20.02	1.00	0.13	6.70
23	150	7597	4.8	0.0	4.8	20.02	1.00	0.13	6.70
24	150	7603	4.8	0.0	4.8	20.02	1.00	0.13	6.70
25	150	7609	4.8	0.0	4.8	20.02	1.00	0.13	6.70
26	150	7615	4.8	0.0	4.8	20.02	1.00	0.13	6.70
27	150	7621	4.8	0.0	4.8	20.02	1.00	0.13	6.70
28	150	7627	4.8	0.0	4.8	20.02	1.00	0.13	6.70
29	150	7633	4.8	0.0	4.8	20.02	1.00	0.13	6.70
30	150	7639	4.8	0.0	4.8	20.02	1.00	0.13	6.70
31	150	7645	4.8	0.0	4.8	20.02	1.00	0.13	6.70
32	150	7651	4.8	0.0	4.8	20.02	1.00	0.13	6.70
33	150	7657	4.8	0.0	4.8	20.02	1.00	0.13	6.70
34	150	7663	4.8	0.0	4.8	20.02	1.00	0.13	6.70
35	150	7669	4.8	0.0	4.8	20.02	1.00	0.13	6.70
36	150	7675	4.8	0.0	4.8	20.02	1.00	0.13	6.70
37	150	7681	4.8	0.0	4.8	20.02	1.00	0.13	6.70
38	150	7687	4.8	0.0	4.8	20.02	1.00	0.13	6.70
39	150	7693	4.8	0.0	4.8	20.02	1.00	0.13	6.70
40	150	7699	4.8	0.0	4.8	20.02	1.00	0.13	6.70
41	150	7705	4.8	0.0	4.8	20.02	1.00	0.13	6.70

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 74+75 to 77+00
Critical Station = 74+75



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 74+75 to 77+00
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	50	ft
h_o =	19.33	ft
TOL =	765.7	ft elevation
Landside =	742.0	ft elevation
Bottom Blanket =	720	ft elevation
blanket =	22.0	ft
z, Landside Toe =	150	ft
t_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	16.91	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	V_w (ft/s)	H_w (ft)
8	152	7930	739.0	0.90	0.29	1.00
9	153	8050	740.0	0.78	0.25	1.00
10	162	8180	740.3	0.81	0.26	1.00
11	155	8330	741.0	1.05	0.33	1.00
12	163	8570	743.5	1.14	0.36	1.01
43	100	7450	746.0	0.82	0.26	1.00
44	100	7470	746.0	0.77	0.24	1.00
45	100	7480	746.0	0.78	0.25	1.00
46	150	7650	742.0	0.91	0.29	1.00
47	150	7700	742.0	0.86	0.27	1.00
48	150	7750	742.0	0.83	0.26	1.00
49	150	7800	742.0	0.82	0.26	1.00
50	150	7850	742.0	0.82	0.26	1.00
51	150	7950	742.0	0.78	0.25	1.00
52	150	8050	742.0	0.71	0.23	1.00
53	150	8140	742.0	0.78	0.25	1.00
54	150	8200	742.0	0.76	0.24	1.00
55	150	8225	742.0	0.81	0.26	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

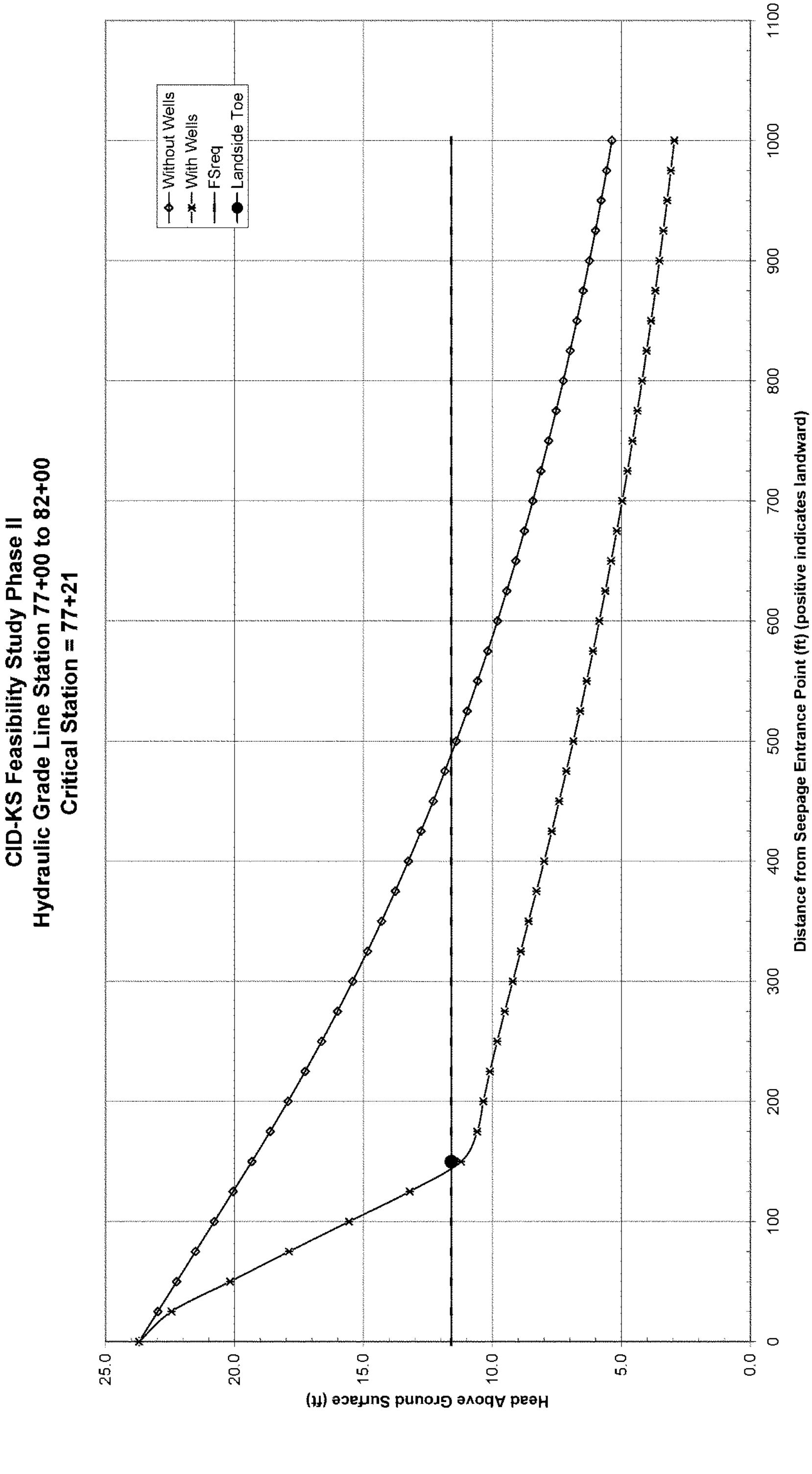
image well locations		
well	x'	y'
8	-152	7930
9	-153	8050
10	-162	8180
11	-155	8330
12	-163	8570
43	-100	7450
44	-100	7470
45	-100	7480
46	-150	7650
47	-150	7700
48	-150	7750
49	-150	7800
50	-150	7850
51	-150	7950
52	-150	8050
53	-150	8140
54	-150	8200
55	-150	8225
0	0	0
0	0	0

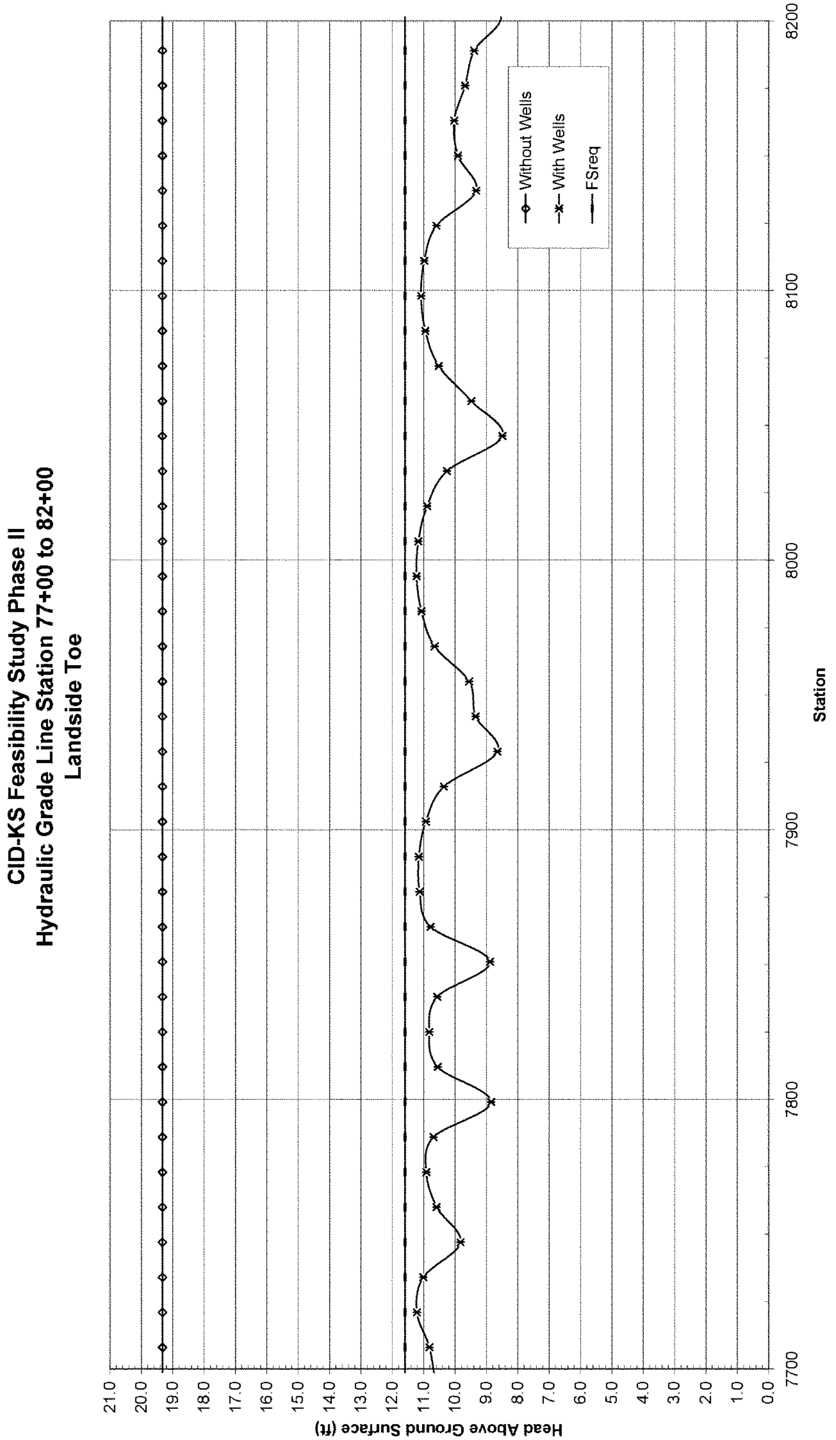
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	7721	23.7	0.0	23.7	11.59	1.00	1.08	0.78
2	25	7721	23.0	1.5	22.4	11.59	1.00	1.02	0.83
3	50	7721	22.2	3.1	20.2	11.59	1.00	0.92	0.92
4	75	7721	21.5	4.6	17.9	11.59	1.00	0.81	1.04
5	100	7721	20.8	6.2	15.6	11.59	1.00	0.71	1.19
6	125	7721	20.1	7.9	13.2	11.59	1.00	0.60	1.40
7	150	7721	19.3	9.1	11.2	11.59	1.00	0.51	1.65
8	175	7721	18.6	9.0	10.6	11.59	1.00	0.48	1.75
9	200	7721	17.9	8.6	10.3	11.59	1.00	0.47	1.79
10	225	7721	17.3	8.2	10.1	11.59	1.00	0.46	1.84
11	250	7721	16.6	7.8	9.8	11.59	1.00	0.45	1.89
12	275	7721	16.0	7.5	9.5	11.59	1.00	0.43	1.95
13	300	7721	15.4	7.2	9.2	11.59	1.00	0.42	2.01
14	325	7721	14.8	6.9	8.9	11.59	1.00	0.40	2.08
15	350	7721	14.3	6.7	8.6	11.59	1.00	0.39	2.16
16	375	7721	13.8	6.5	8.3	11.59	1.00	0.38	2.24
17	400	7721	13.3	6.3	8.0	11.59	1.00	0.36	2.32
18	425	7721	12.8	6.1	7.7	11.59	1.00	0.35	2.41
19	450	7721	12.3	5.9	7.4	11.59	1.00	0.34	2.50
20	475	7721	11.8	5.7	7.1	11.59	1.00	0.32	2.60
21	500	7721	11.4	5.5	6.9	11.59	1.00	0.31	2.70
22	525	7721	11.0	5.4	6.6	11.59	1.00	0.30	2.81
23	550	7721	10.6	5.2	6.3	11.59	1.00	0.29	2.92
24	575	7721	10.2	5.1	6.1	11.59	1.00	0.28	3.04
25	600	7721	9.8	5.0	5.9	11.59	1.00	0.27	3.17
26	625	7721	9.4	4.8	5.6	11.59	1.00	0.26	3.30
27	650	7721	9.1	4.7	5.4	11.59	1.00	0.25	3.43
28	675	7721	8.8	4.6	5.2	11.59	1.00	0.24	3.58
29	700	7721	8.4	4.5	5.0	11.59	1.00	0.23	3.73
30	725	7721	8.1	4.4	4.8	11.59	1.00	0.22	3.89
31	750	7721	7.8	4.3	4.6	11.59	1.00	0.21	4.06
32	775	7721	7.5	4.2	4.4	11.59	1.00	0.20	4.23
33	800	7721	7.3	4.1	4.2	11.59	1.00	0.19	4.42
34	825	7721	7.0	4.0	4.0	11.59	1.00	0.18	4.61
35	850	7721	6.7	3.9	3.8	11.59	1.00	0.17	4.82
36	875	7721	6.5	3.8	3.7	11.59	1.00	0.17	5.03
37	900	7721	6.2	3.7	3.5	11.59	1.00	0.16	5.26
38	925	7721	6.0	3.6	3.4	11.59	1.00	0.15	5.50
39	950	7721	5.8	3.6	3.2	11.59	1.00	0.15	5.75
40	975	7721	5.6	3.5	3.1	11.59	1.00	0.14	6.01
41	1000	7721	5.4	3.4	2.9	11.59	1.00	0.13	6.29

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	150	7695	19.3	9.7	10.6	11.59	1.00	0.48	1.74
2	150	7708	19.3	9.5	10.8	11.59	1.00	0.49	1.71
3	150	7721	19.3	9.1	11.2	11.59	1.00	0.51	1.65
4	150	7734	19.3	9.3	11.0	11.59	1.00	0.50	1.68
5	150	7747	19.3	10.5	9.8	11.59	1.00	0.45	1.89
6	150	7760	19.3	9.7	10.6	11.59	1.00	0.48	1.75
7	150	7773	19.3	9.4	10.9	11.59	1.00	0.50	1.70
8	150	7786	19.3	9.6	10.7	11.59	1.00	0.49	1.73
9	150	7799	19.3	11.5	8.8	11.59	1.00	0.40	2.10
10	150	7812	19.3	9.8	10.6	11.59	1.00	0.48	1.76
11	150	7825	19.3	9.5	10.8	11.59	1.00	0.49	1.71
12	150	7838	19.3	9.8	10.6	11.59	1.00	0.48	1.76
13	150	7851	19.3	11.4	8.9	11.59	1.00	0.40	2.09
14	150	7864	19.3	9.6	10.8	11.59	1.00	0.49	1.72
15	150	7877	19.3	9.2	11.1	11.59	1.00	0.51	1.67
16	150	7890	19.3	9.2	11.2	11.59	1.00	0.51	1.66
17	150	7903	19.3	9.4	10.9	11.59	1.00	0.50	1.70
18	150	7916	19.3	10.0	10.4	11.59	1.00	0.47	1.79
19	150	7929	19.3	11.7	8.6	11.59	1.00	0.39	2.15
20	150	7942	19.3	11.0	9.3	11.59	1.00	0.42	1.98
21	150	7955	19.3	10.8	9.5	11.59	1.00	0.43	1.94
22	150	7968	19.3	9.7	10.7	11.59	1.00	0.48	1.74
23	150	7981	19.3	9.3	11.1	11.59	1.00	0.50	1.67
24	150	7994	19.3	9.1	11.2	11.59	1.00	0.51	1.65
25	150	8007	19.3	9.2	11.2	11.59	1.00	0.51	1.66
26	150	8020	19.3	9.4	10.9	11.59	1.00	0.49	1.70
27	150	8033	19.3	10.1	10.3	11.59	1.00	0.47	1.81
28	150	8046	19.3	11.8	8.5	11.59	1.00	0.39	2.18
29	150	8059	19.3	10.9	9.5	11.59	1.00	0.43	1.96
30	150	8072	19.3	9.8	10.5	11.59	1.00	0.48	1.76
31	150	8085	19.3	9.4	10.9	11.59	1.00	0.50	1.69
32	150	8098	19.3	9.2	11.1	11.59	1.00	0.50	1.67
33	150	8111	19.3	9.3	11.0	11.59	1.00	0.50	1.69
34	150	8124	19.3	9.7	10.6	11.59	1.00	0.48	1.75
35	150	8137	19.3	11.0	9.3	11.59	1.00	0.42	1.99
36	150	8150	19.3	10.4	9.9	11.59	1.00	0.45	1.87
37	150	8163	19.3	10.3	10.0	11.59	1.00	0.46	1.85
38	150	8176	19.3	10.7	9.7	11.59	1.00	0.44	1.92
39	150	8189	19.3	10.9	9.4	11.59	1.00	0.43	1.97
40	150	8202	19.3	11.8	8.5	11.59	1.00	0.39	2.17
41	150	8215	19.3	10.6	9.8	11.59	1.00	0.44	1.90


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 77+00 to 82+00
Critical Station = 77+21
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
25.0
20.0
15.0
10.0
5.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 77+00 to 82+00
Landside Toe
Head Above Ground Surface (ft)
21.0
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
7700
7800
7900
8000
8100
8200
Station
Without Wells
With Wells
FSreq

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	50	ft
h_o =	21.02	ft
TOL =	766	ft elevation
Landside =	740.0	ft elevation
Bottom Blanket =	720	ft elevation
blanket =	20.0	ft
z, Landside Toe =	150	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	17.09	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
9	153	8050	740.0	0.77	0.24	1.00
10	162	8180	740.3	0.66	0.21	1.00
11	155	8330	741.0	0.57	0.18	1.00
12	163	8570	743.5	0.65	0.21	1.00
13	169	8776	745.0	0.89	0.28	1.00
51	150	7950	742.0	0.95	0.30	1.00
52	150	8050	742.0	0.70	0.22	1.00
53	150	8140	742.0	0.68	0.22	1.00
54	150	8200	742.0	0.59	0.19	1.00
55	150	8225	742.0	0.58	0.18	1.00
56	150	8250	740.0	0.63	0.20	1.00
57	150	8275	740.0	0.64	0.20	1.00
58	150	8325	740.0	0.60	0.19	1.00
59	150	8350	740.0	0.62	0.20	1.00
60	150	8390	740.0	0.66	0.21	1.00
61	150	8430	740.0	0.67	0.21	1.00
62	150	8460	740.0	0.68	0.22	1.00
63	150	8490	740.0	0.69	0.22	1.00
64	150	8520	740.0	0.71	0.23	1.00
65	150	8550	740.0	0.75	0.24	1.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

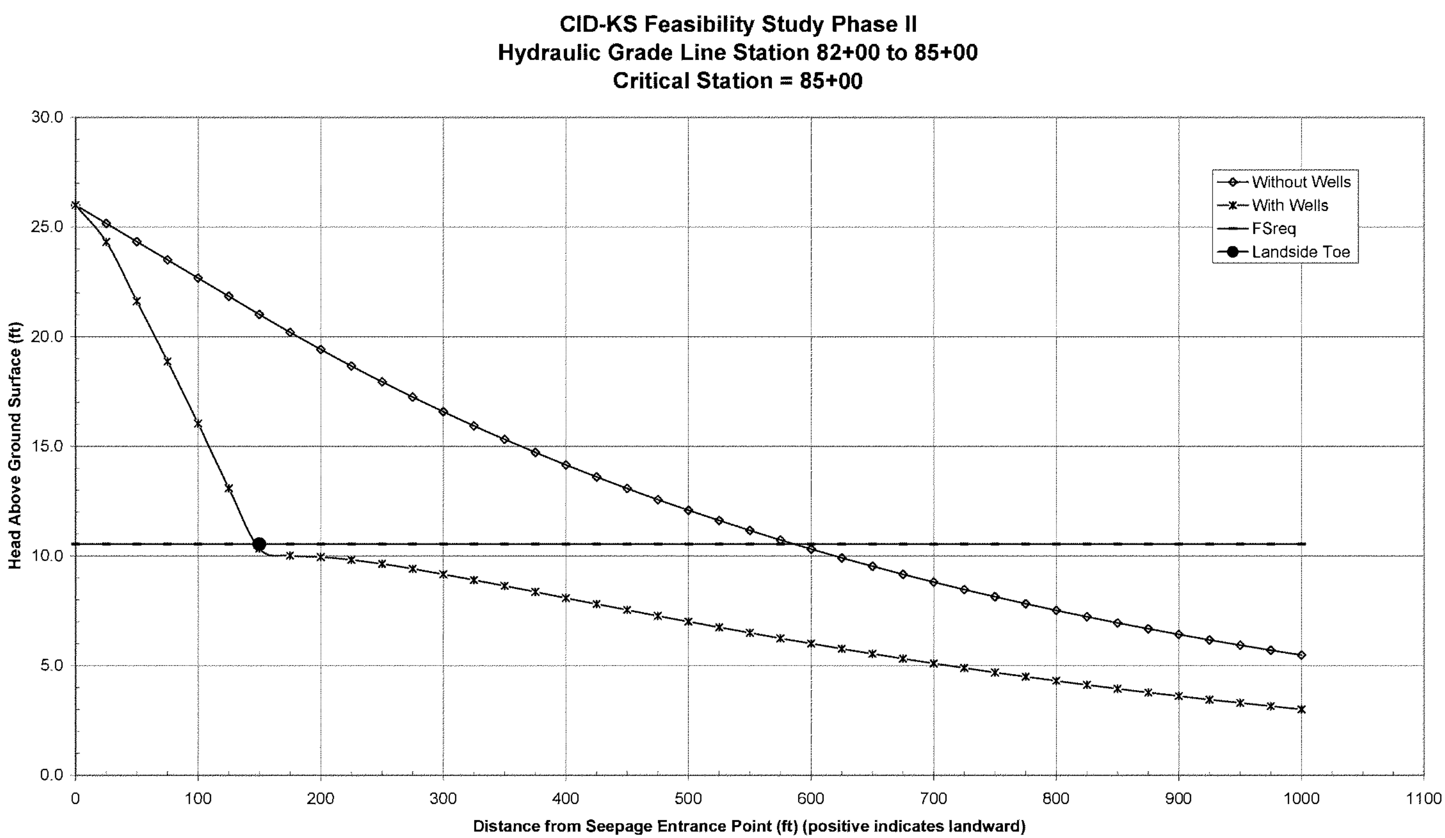
image well locations		
well	x'	y'
9	-153	8050
10	-162	8180
11	-155	8330
12	-163	8570
13	-169	8776
51	-150	7950
52	-150	8050
53	-150	8140
54	-150	8200
55	-150	8225
56	-150	8250
57	-150	8275
58	-150	8325
59	-150	8350
60	-150	8390
61	-150	8430
62	-150	8460
63	-150	8490
64	-150	8520
65	-150	8550

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

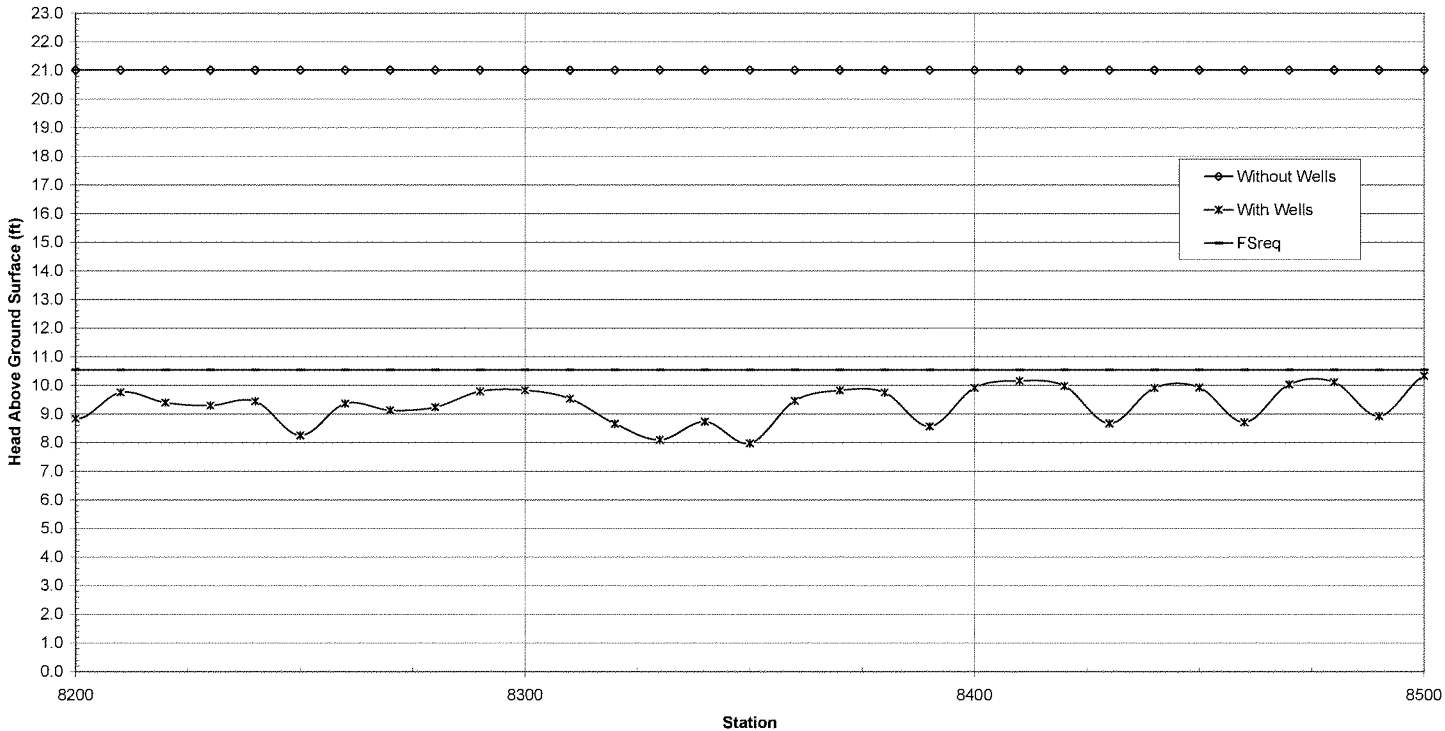
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	8500	26.0	0.0	26.0	10.54	1.00	1.30	0.65
2	25	8500	25.2	1.8	24.3	10.54	1.00	1.22	0.69
3	50	8500	24.3	3.7	21.6	10.54	1.00	1.08	0.78
4	75	8500	23.5	5.6	18.9	10.54	1.00	0.94	0.89
5	100	8500	22.7	7.6	16.0	10.54	1.00	0.80	1.05
6	125	8500	21.8	9.8	13.1	10.54	1.00	0.65	1.29
7	150	8500	21.0	11.7	10.3	10.54	1.00	0.52	1.63
8	175	8500	20.2	11.2	10.0	10.54	1.00	0.50	1.68
9	200	8500	19.4	10.5	10.0	10.54	1.00	0.50	1.69
10	225	8500	18.7	9.8	9.8	10.54	1.00	0.49	1.72
11	250	8500	17.9	9.3	9.6	10.54	1.00	0.48	1.75
12	275	8500	17.2	8.6	9.4	10.54	1.00	0.47	1.79
13	300	8500	16.6	8.4	9.2	10.54	1.00	0.46	1.84
14	325	8500	15.9	8.0	8.9	10.54	1.00	0.45	1.89
15	350	8500	15.3	7.7	8.6	10.54	1.00	0.43	1.95
16	375	8500	14.7	7.4	8.4	10.54	1.00	0.42	2.02
17	400	8500	14.2	7.1	8.1	10.54	1.00	0.40	2.09
18	425	8500	13.6	6.8	7.8	10.54	1.00	0.39	2.16
19	450	8500	13.1	6.5	7.5	10.54	1.00	0.38	2.24
20	475	8500	12.6	6.3	7.3	10.54	1.00	0.36	2.32
21	500	8500	12.1	6.1	7.0	10.54	1.00	0.35	2.41
22	525	8500	11.6	5.9	6.7	10.54	1.00	0.34	2.50
23	550	8500	11.2	5.7	6.5	10.54	1.00	0.32	2.60
24	575	8500	10.7	5.5	6.2	10.54	1.00	0.31	2.70
25	600	8500	10.3	5.3	6.0	10.54	1.00	0.30	2.81
26	625	8500	9.9	5.1	5.8	10.54	1.00	0.29	2.92
27	650	8500	9.5	5.0	5.5	10.54	1.00	0.28	3.04
28	675	8500	9.2	4.8	5.3	10.54	1.00	0.27	3.17
29	700	8500	8.8	4.7	5.1	10.54	1.00	0.26	3.30
30	725	8500	8.5	4.6	4.9	10.54	1.00	0.24	3.45
31	750	8500	8.1	4.4	4.7	10.54	1.00	0.23	3.59
32	775	8500	7.8	4.3	4.5	10.54	1.00	0.22	3.75
33	800	8500	7.5	4.2	4.3	10.54	1.00	0.22	3.92
34	825	8500	7.2	4.1	4.1	10.54	1.00	0.21	4.09
35	850	8500	6.9	4.0	3.9	10.54	1.00	0.20	4.27
36	875	8500	6.7	3.9	3.8	10.54	1.00	0.19	4.47
37	900	8500	6.4	3.8	3.6	10.54	1.00	0.18	4.67
38	925	8500	6.2	3.7	3.4	10.54	1.00	0.17	4.89
39	950	8500	5.9	3.6	3.3	10.54	1.00	0.16	5.11
40	975	8500	5.7	3.6	3.1	10.54	1.00	0.16	5.35
41	1000	8500	5.5	3.5	3.0	10.54	1.00	0.15	5.61

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	150	8190	21.0	11.9	10.1	10.54	1.00	0.51	1.66
2	150	8200	21.0	13.2	8.8	10.54	1.00	0.44	1.91
3	150	8210	21.0	12.3	9.8	10.54	1.00	0.49	1.73
4	150	8220	21.0	12.6	9.4	10.54	1.00	0.47	1.79
5	150	8230	21.0	12.7	9.3	10.54	1.00	0.46	1.81
6	150	8240	21.0	12.6	9.4	10.54	1.00	0.47	1.79
7	150	8250	21.0	13.8	8.2	10.54	1.00	0.41	2.04
8	150	8260	21.0	12.7	9.4	10.54	1.00	0.47	1.80
9	150	8270	21.0	12.9	9.1	10.54	1.00	0.46	1.85
10	150	8280	21.0	12.8	9.2	10.54	1.00	0.46	1.82
11	150	8290	21.0	12.2	9.8	10.54	1.00	0.49	1.72
12	150	8300	21.0	12.2	9.8	10.54	1.00	0.49	1.72
13	150	8310	21.0	12.5	9.5	10.54	1.00	0.48	1.77
14	150	8320	21.0	13.4	8.7	10.54	1.00	0.43	1.95
15	150	8330	21.0	13.9	8.1	10.54	1.00	0.40	2.08
16	150	8340	21.0	13.3	8.7	10.54	1.00	0.44	1.93
17	150	8350	21.0	14.0	8.0	10.54	1.00	0.40	2.11
18	150	8360	21.0	12.6	9.5	10.54	1.00	0.47	1.78
19	150	8370	21.0	12.2	9.8	10.54	1.00	0.49	1.72
20	150	8380	21.0	12.3	9.7	10.54	1.00	0.49	1.73
21	150	8390	21.0	13.4	8.6	10.54	1.00	0.43	1.97
22	150	8400	21.0	12.1	9.9	10.54	1.00	0.50	1.70
23	150	8410	21.0	11.9	10.2	10.54	1.00	0.51	1.66
24	150	8420	21.0	12.0	10.0	10.54	1.00	0.50	1.69
25	150	8430	21.0	13.3	8.7	10.54	1.00	0.43	1.95
26	150	8440	21.0	12.1	9.9	10.54	1.00	0.50	1.70
27	150	8450	21.0	12.1	9.9	10.54	1.00	0.50	1.70
28	150	8460	21.0	13.3	8.7	10.54	1.00	0.44	1.94
29	150	8470	21.0	12.0	10.0	10.54	1.00	0.50	1.68
30	150	8480	21.0	11.9	10.1	10.54	1.00	0.51	1.67
31	150	8490	21.0	13.1	8.9	10.54	1.00	0.45	1.89
32	150	8500	21.0	11.7	10.3	10.54	1.00	0.52	1.63
33	150	8510	21.0	11.5	10.5	10.54	1.00	0.52	1.61
34	150	8520	21.0	12.7	9.3	10.54	1.00	0.47	1.81
35	150	8530	21.0	11.2	10.8	10.54	1.00	0.54	1.56
36	150	8540	21.0	11.0	11.1	10.54	1.00	0.55	1.52
37	150	8550	21.0	12.1	9.9	10.54	1.00	0.50	1.70
38	150	8560	21.0	10.3	11.7	10.54	1.00	0.59	1.44
39	150	8570	21.0	9.8	12.5	10.54	1.00	0.62	1.35
40	150	8580	21.0	8.8	13.2	10.54	1.00	0.66	1.26
41	150	8590	21.0	8.1	13.9	10.54	1.00	0.69	1.21


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 82+00 to 85+00
Critical Station = 85+00
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
30.0
25.0
20.0
15.0
10.0
5.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100
Distance from Seepage Entrance Point (ft) (positive indicates landward)

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 82+00 to 85+00
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	50	ft
h_o =	19.82	ft
TOL =	766.3	ft elevation
Landside =	742.0	ft elevation
Bottom Blanket =	720	ft elevation
blanket =	22.0	ft
z, Landside Toe =	150	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	16.47	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
11	155	8330	741.0	0.69	0.22	1.00
12	163	8570	743.5	0.60	0.19	1.00
13	169	8776	745.0	0.62	0.20	1.00
14	162	8980	747.0	0.82	0.26	1.00
57	150	8275	740.0	0.91	0.29	1.00
58	150	8325	740.0	0.74	0.23	1.00
59	150	8350	740.0	0.74	0.23	1.00
60	150	8390	740.0	0.74	0.24	1.00
61	150	8430	740.0	0.72	0.23	1.00
62	150	8460	740.0	0.71	0.23	1.00
63	150	8490	740.0	0.71	0.22	1.00
64	150	8520	740.0	0.71	0.23	1.00
65	150	8550	740.0	0.71	0.23	1.00
66	150	8625	742.0	0.71	0.23	1.00
67	150	8675	742.0	0.74	0.23	1.00
68	150	8730	742.0	0.75	0.24	1.00
69	150	8780	742.0	0.75	0.24	1.00
70	150	8820	742.0	0.83	0.26	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

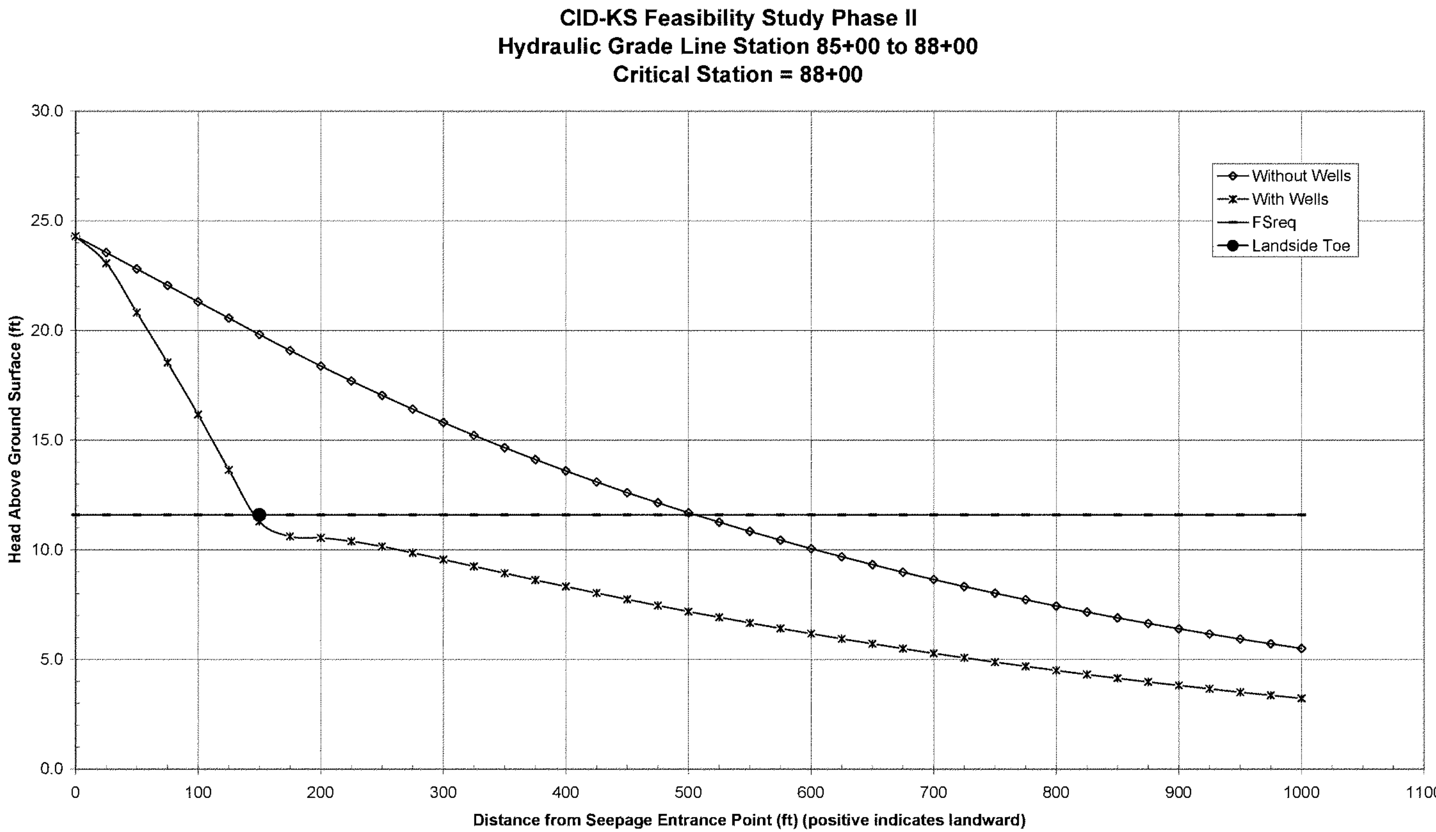
image well locations		
well	x'	y'
11	-155	8330
12	-163	8570
13	-169	8776
14	-162	8960
57	-150	8275
58	-150	8325
59	-150	8350
60	-150	8390
61	-150	8430
62	-150	8460
63	-150	8490
64	-150	8520
65	-150	8550
66	-150	8625
67	-150	8675
68	-150	8730
69	-150	8780
70	-150	8820
0	0	0
0	0	0

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

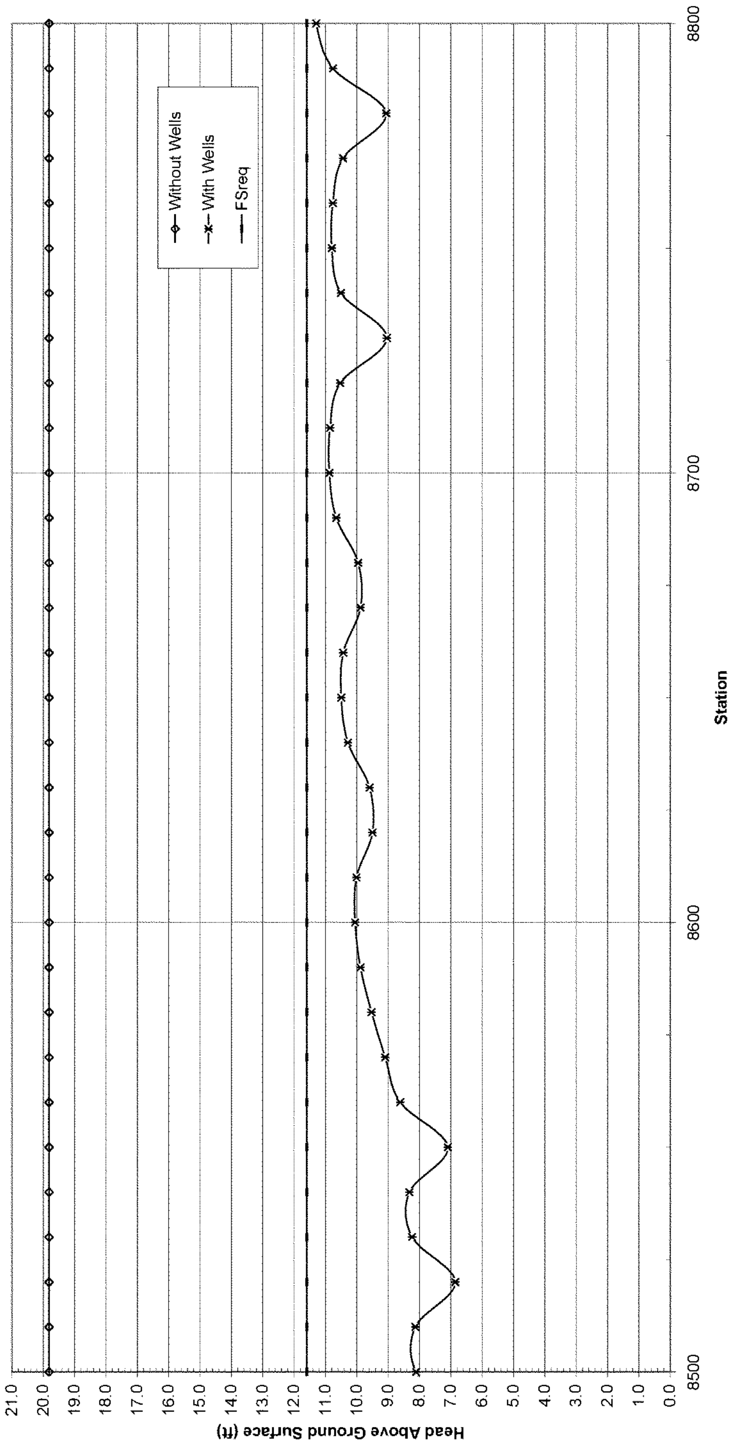
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	8800	24.3	0.0	24.3	11.59	1.00	1.10	0.76
2	25	8800	23.6	1.5	23.1	11.59	1.00	1.05	0.80
3	50	8800	22.8	3.0	20.8	11.59	1.00	0.95	0.89
4	75	8800	22.1	4.5	18.5	11.59	1.00	0.84	1.00
5	100	8800	21.3	6.1	16.2	11.59	1.00	0.73	1.15
6	125	8800	20.6	7.9	13.8	11.59	1.00	0.62	1.36
7	150	8800	19.6	9.5	11.3	11.59	1.00	0.51	1.64
8	175	8800	19.1	9.5	10.6	11.59	1.00	0.48	1.75
9	200	8800	18.4	8.8	10.5	11.59	1.00	0.48	1.76
10	225	8800	17.7	8.3	10.4	11.59	1.00	0.47	1.78
11	250	8800	17.0	7.9	10.1	11.59	1.00	0.46	1.83
12	275	8800	16.4	7.6	9.9	11.59	1.00	0.45	1.88
13	300	8800	15.8	7.2	9.6	11.59	1.00	0.43	1.94
14	325	8800	15.2	7.0	9.2	11.59	1.00	0.42	2.01
15	350	8800	14.7	6.7	8.9	11.59	1.00	0.41	2.08
16	375	8800	14.1	6.5	8.6	11.59	1.00	0.39	2.15
17	400	8800	13.6	6.3	8.3	11.59	1.00	0.38	2.23
18	425	8800	13.1	6.1	8.0	11.59	1.00	0.36	2.31
19	450	8800	12.6	5.9	7.7	11.59	1.00	0.35	2.40
20	475	8800	12.1	5.7	7.5	11.59	1.00	0.34	2.49
21	500	8800	11.7	5.5	7.2	11.59	1.00	0.33	2.58
22	525	8800	11.3	5.3	6.9	11.59	1.00	0.31	2.68
23	550	8800	10.8	5.2	6.7	11.59	1.00	0.30	2.78
24	575	8800	10.4	5.0	6.4	11.59	1.00	0.29	2.89
25	600	8800	10.1	4.9	6.2	11.59	1.00	0.28	3.00
26	625	8800	9.7	4.7	5.9	11.59	1.00	0.27	3.12
27	650	8800	9.3	4.6	5.7	11.59	1.00	0.26	3.25
28	675	8800	9.0	4.5	5.5	11.59	1.00	0.25	3.38
29	700	8800	8.6	4.4	5.3	11.59	1.00	0.24	3.51
30	725	8800	8.3	4.3	5.1	11.59	1.00	0.23	3.65
31	750	8800	8.0	4.1	4.9	11.59	1.00	0.22	3.80
32	775	8800	7.7	4.0	4.7	11.59	1.00	0.21	3.96
33	800	8800	7.4	3.9	4.5	11.59	1.00	0.20	4.12
34	825	8800	7.2	3.8	4.3	11.59	1.00	0.20	4.29
35	850	8800	6.9	3.8	4.1	11.59	1.00	0.19	4.47
36	875	8800	6.6	3.7	4.0	11.59	1.00	0.18	4.66
37	900	8800	6.4	3.6	3.8	11.59	1.00	0.17	4.86
38	925	8800	6.2	3.5	3.7	11.59	1.00	0.17	5.07
39	950	8800	5.9	3.4	3.5	11.59	1.00	0.16	5.28
40	975	8800	5.7	3.3	3.4	11.59	1.00	0.15	5.51
41	1000	8800	5.5	3.3	3.2	11.59	1.00	0.15	5.75

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	150	8490	19.8	14.0	6.8	11.59	1.00	0.31	2.74
2	150	8500	19.8	12.7	8.1	11.59	1.00	0.37	2.29
3	150	8510	19.8	12.7	8.1	11.59	1.00	0.37	2.28
4	150	8520	19.8	14.0	6.9	11.59	1.00	0.31	2.70
5	150	8530	19.8	12.6	8.2	11.59	1.00	0.37	2.25
6	150	8540	19.8	12.5	8.3	11.59	1.00	0.38	2.23
7	150	8550	19.8	13.7	7.1	11.59	1.00	0.32	2.61
8	150	8560	19.8	12.2	8.6	11.59	1.00	0.39	2.15
9	150	8570	19.8	11.7	9.1	11.59	1.00	0.41	2.04
10	150	8580	19.8	11.3	9.5	11.59	1.00	0.43	1.95
11	150	8590	19.8	10.9	9.9	11.59	1.00	0.45	1.88
12	150	8600	19.8	10.8	10.1	11.59	1.00	0.46	1.84
13	150	8610	19.8	10.8	10.0	11.59	1.00	0.46	1.85
14	150	8620	19.8	11.3	9.5	11.59	1.00	0.43	1.95
15	150	8630	19.8	11.2	9.6	11.59	1.00	0.44	1.93
16	150	8640	19.8	10.5	10.3	11.59	1.00	0.47	1.80
17	150	8650	19.8	10.3	10.5	11.59	1.00	0.48	1.77
18	150	8660	19.8	10.4	10.4	11.59	1.00	0.47	1.78
19	150	8670	19.8	10.9	9.9	11.59	1.00	0.45	1.88
20	150	8680	19.8	10.9	10.0	11.59	1.00	0.45	1.86
21	150	8690	19.9	10.2	10.7	11.59	1.00	0.48	1.74
22	150	8700	19.8	9.9	10.9	11.59	1.00	0.49	1.71
23	150	8710	19.8	10.0	10.9	11.59	1.00	0.49	1.71
24	150	8720	19.8	10.3	10.5	11.59	1.00	0.48	1.76
25	150	8730	19.8	11.6	9.0	11.59	1.00	0.41	2.05
26	150	8740	19.8	10.3	10.5	11.59	1.00	0.48	1.77
27	150	8750	19.8	10.0	10.8	11.59	1.00	0.49	1.72
28	150	8760	19.8	10.1	10.8	11.59	1.00	0.49	1.72
29	150	8770	19.8	10.4	10.4	11.59	1.00	0.47	1.78
30	150	8780	19.8	11.8	9.1	11.59	1.00	0.41	2.05
31	150	8790	19.8	10.1	10.8	11.59	1.00	0.49	1.72
32	150	8800	19.8	9.5	11.3	11.59	1.00	0.51	1.64
33	150	8810	19.9	9.4	11.4	11.59	1.00	0.52	1.63
34	150	8820	19.8	10.6	10.2	11.59	1.00	0.46	1.82
35	150	8830	19.8	8.6	12.3	11.59	1.00	0.56	1.51
36	150	8840	19.8	7.7	13.1	11.59	1.00	0.60	1.42
37	150	8850	19.8	7.1	13.7	11.59	1.00	0.62	1.36
38	150	8860	19.8	6.7	14.1	11.59	1.00	0.64	1.31
39	150	8870	19.8	6.3	14.5	11.59	1.00	0.66	1.28
40	150	8880	19.8	6.0	14.8	11.59	1.00	0.67	1.25
41	150	8890	19.8	5.7	15.1	11.59	1.00	0.69	1.23


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 85+00 to 88+00
Critical Station = 88+00
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
30.0
25.0
20.0
15.0
10.0
5.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100
Distance from Seepage Entrance Point (ft) (positive indicates landward)

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 85+00 to 88+00
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	50	ft
h_B =	17.74	ft
TOL =	786.5	ft elevation
Landside =	745.0	ft elevation
Bottom Blanket =	720	ft elevation
blanket =	25.0	ft
z, Landside Toe =	150	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	11.17	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
12	163	8570	743.5	0.99	0.32	1.00
13	169	8776	745.0	0.71	0.23	1.00
14	162	8980	747.0	0.80	0.25	1.00
15	160	9200	748.0	0.96	0.31	1.00
66	150	8625	742.0	0.99	0.31	1.00
67	150	8675	742.0	0.93	0.30	1.00
68	150	8730	742.0	0.89	0.28	1.00
69	150	8780	742.0	0.85	0.27	1.00
70	150	8820	742.0	0.92	0.29	1.00
71	150	8950	745.0	0.89	0.28	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

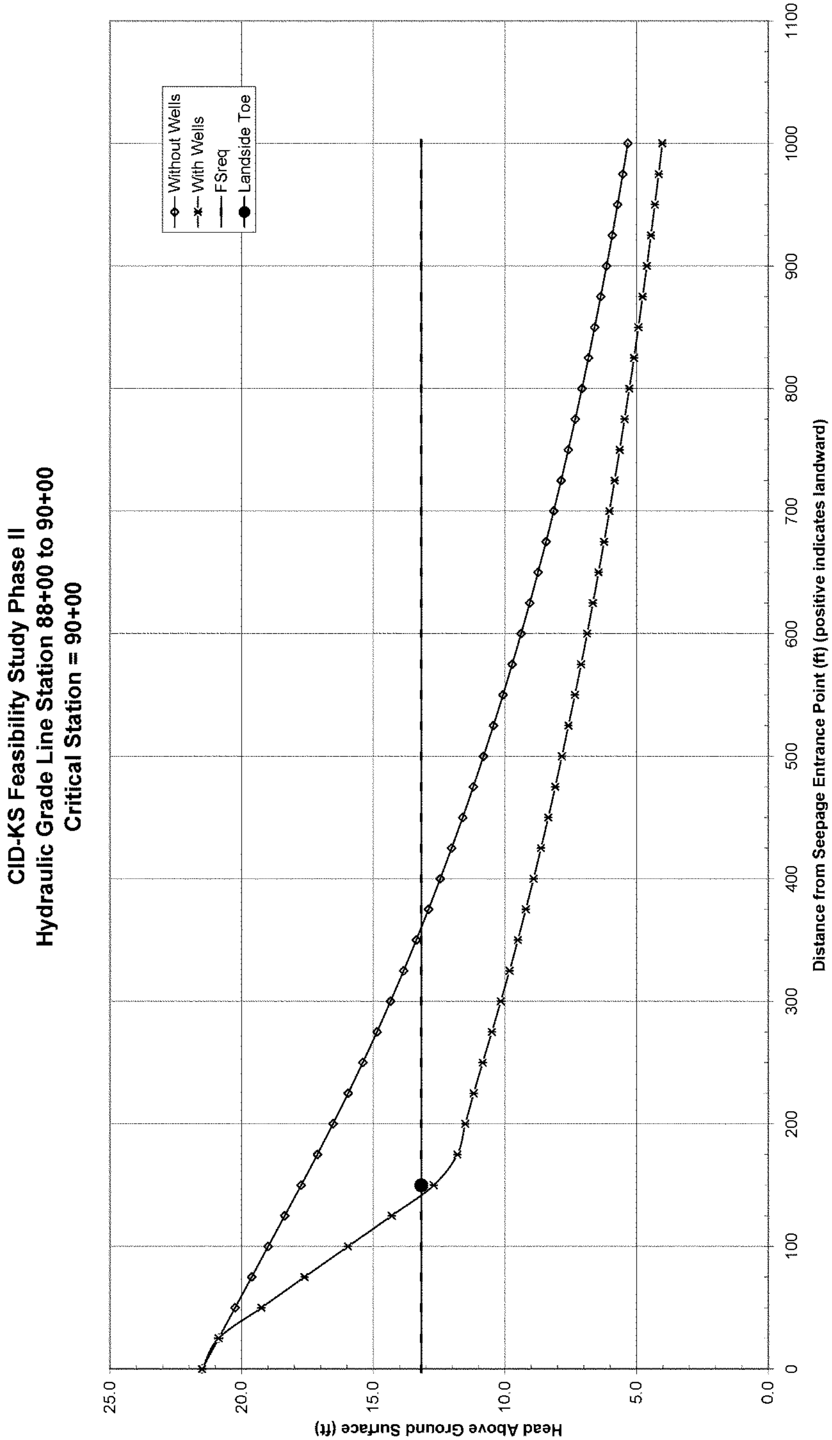
image well locations		
well	x'	y'
12	-163	8570
13	-169	8776
14	-162	8980
15	-160	9200
66	-150	8625
67	-150	8675
68	-150	8730
69	-150	8780
70	-150	8820
71	-150	8950
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	9000	21.5	0.0	21.5	13.17	1.00	0.86	0.98
2	25	9000	20.9	1.0	20.9	13.17	1.00	0.83	1.01
3	50	9000	20.2	2.0	19.2	13.17	1.00	0.77	1.09
4	75	9000	19.6	3.0	17.6	13.17	1.00	0.70	1.20
5	100	9000	19.0	4.0	16.0	13.17	1.00	0.64	1.32
6	125	9000	18.4	5.1	14.3	13.17	1.00	0.57	1.47
7	150	9000	17.7	6.0	12.7	13.17	1.00	0.51	1.66
8	175	9000	17.1	6.3	11.8	13.17	1.00	0.47	1.78
9	200	9000	16.5	6.0	11.5	13.17	1.00	0.46	1.83
10	225	9000	16.0	5.8	11.2	13.17	1.00	0.45	1.88
11	250	9000	15.4	5.6	10.8	13.17	1.00	0.43	1.94
12	275	9000	14.9	5.4	10.5	13.17	1.00	0.42	2.01
13	300	9000	14.3	5.2	10.2	13.17	1.00	0.41	2.08
14	325	9000	13.8	5.0	9.8	13.17	1.00	0.39	2.14
15	350	9000	13.4	4.9	9.5	13.17	1.00	0.38	2.22
16	375	9000	12.9	4.7	9.2	13.17	1.00	0.37	2.29
17	400	9000	12.5	4.5	8.9	13.17	1.00	0.36	2.36
18	425	9000	12.0	4.4	8.6	13.17	1.00	0.35	2.44
19	450	9000	11.6	4.2	8.4	13.17	1.00	0.33	2.52
20	475	9000	11.2	4.1	8.1	13.17	1.00	0.32	2.60
21	500	9000	10.8	4.0	7.8	13.17	1.00	0.31	2.69
22	525	9000	10.4	3.9	7.6	13.17	1.00	0.30	2.78
23	550	9000	10.1	3.7	7.3	13.17	1.00	0.29	2.87
24	575	9000	9.7	3.6	7.1	13.17	1.00	0.28	2.96
25	600	9000	9.4	3.5	6.9	13.17	1.00	0.28	3.06
26	625	9000	9.1	3.4	6.7	13.17	1.00	0.27	3.17
27	650	9000	8.7	3.3	6.4	13.17	1.00	0.26	3.27
28	675	9000	8.4	3.2	6.2	13.17	1.00	0.25	3.38
29	700	9000	8.1	3.1	6.0	13.17	1.00	0.24	3.49
30	725	9000	7.9	3.0	5.8	13.17	1.00	0.23	3.61
31	750	9000	7.6	3.0	5.6	13.17	1.00	0.23	3.74
32	775	9000	7.3	2.9	5.5	13.17	1.00	0.22	3.86
33	800	9000	7.1	2.8	5.3	13.17	1.00	0.21	3.99
34	825	9000	6.8	2.7	5.1	13.17	1.00	0.20	4.13
35	850	9000	6.6	2.7	4.9	13.17	1.00	0.20	4.27
36	875	9000	6.4	2.6	4.8	13.17	1.00	0.19	4.42
37	900	9000	6.1	2.5	4.6	13.17	1.00	0.18	4.57
38	925	9000	5.9	2.5	4.5	13.17	1.00	0.18	4.73
39	950	9000	5.7	2.4	4.3	13.17	1.00	0.17	4.89
40	975	9000	5.5	2.4	4.2	13.17	1.00	0.17	5.06
41	1000	9000	5.3	2.3	4.0	13.17	1.00	0.16	5.24

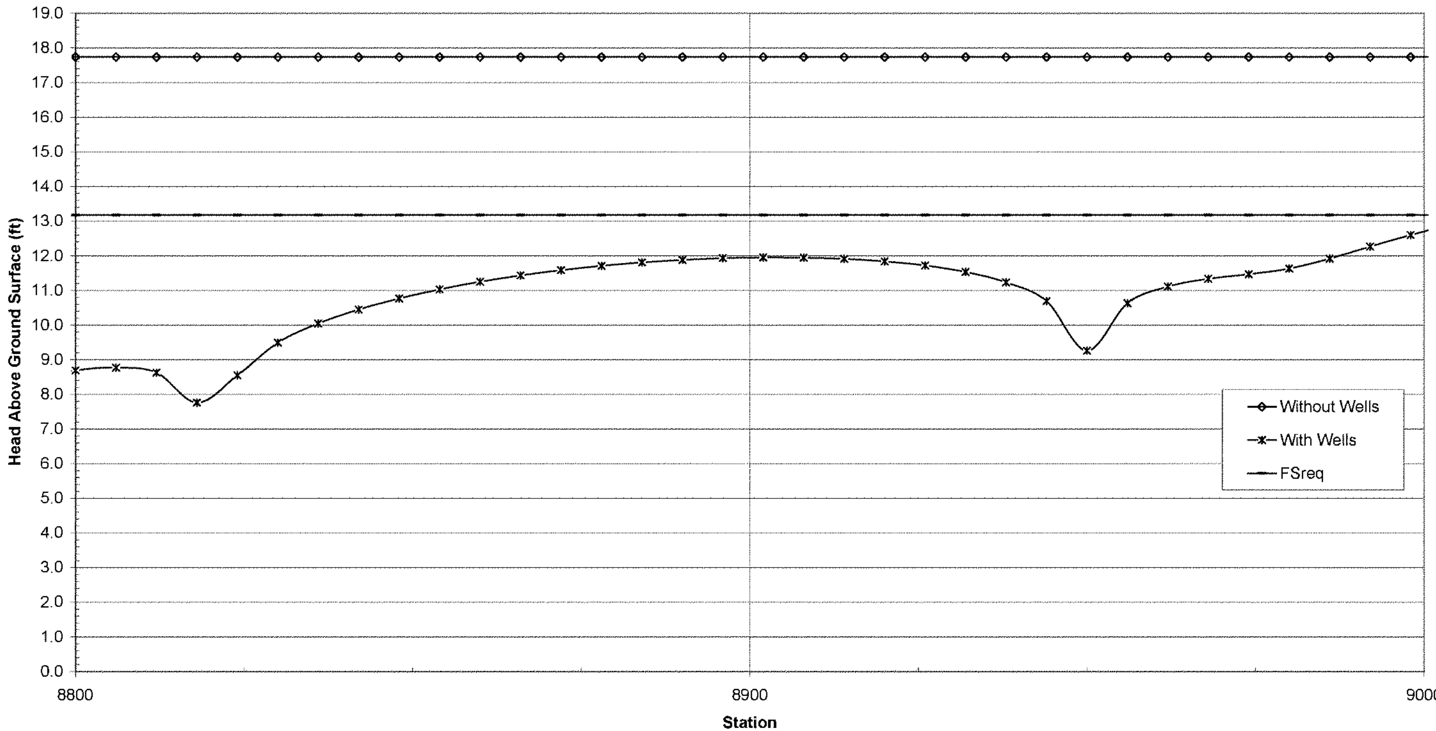
Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	150	8794	17.7	10.3	8.4	13.17	1.00	0.34	2.50
2	150	8800	17.7	10.0	8.7	13.17	1.00	0.35	2.42
3	150	8806	17.7	10.0	8.8	13.17	1.00	0.35	2.40
4	150	8812	17.7	10.1	8.6	13.17	1.00	0.35	2.44
5	150	8818	17.7	11.0	7.8	13.17	1.00	0.31	2.71
6	150	8824	17.7	10.2	8.6	13.17	1.00	0.34	2.46
7	150	8830	17.7	9.2	9.5	13.17	1.00	0.38	2.22
8	150	8836	17.7	8.7	10.0	13.17	1.00	0.40	2.10
9	150	8842	17.7	8.3	10.4	13.17	1.00	0.42	2.02
10	150	8848	17.7	8.0	10.8	13.17	1.00	0.43	1.96
11	150	8854	17.7	7.7	11.0	13.17	1.00	0.44	1.91
12	150	8860	17.7	7.5	11.3	13.17	1.00	0.45	1.87
13	150	8866	17.7	7.3	11.4	13.17	1.00	0.46	1.84
14	150	8872	17.7	7.2	11.6	13.17	1.00	0.46	1.82
15	150	8878	17.7	7.0	11.7	13.17	1.00	0.47	1.80
16	150	8884	17.7	6.9	11.8	13.17	1.00	0.47	1.78
17	150	8890	17.7	6.9	11.9	13.17	1.00	0.48	1.77
18	150	8896	17.7	6.8	11.9	13.17	1.00	0.48	1.77
19	150	8902	17.7	6.8	12.0	13.17	1.00	0.48	1.76
20	150	8908	17.7	6.8	11.9	13.17	1.00	0.48	1.76
21	150	8914	17.7	6.8	11.9	13.17	1.00	0.48	1.77
22	150	8920	17.7	6.9	11.8	13.17	1.00	0.47	1.78
23	150	8926	17.7	7.0	11.7	13.17	1.00	0.47	1.80
24	150	8932	17.7	7.2	11.5	13.17	1.00	0.46	1.83
25	150	8938	17.7	7.5	11.2	13.17	1.00	0.45	1.88
26	150	8944	17.7	8.0	10.7	13.17	1.00	0.43	1.97
27	150	8950	17.7	9.5	9.3	13.17	1.00	0.37	2.28
28	150	8956	17.7	8.1	10.6	13.17	1.00	0.43	1.98
29	150	8962	17.7	7.6	11.1	13.17	1.00	0.44	1.90
30	150	8968	17.7	7.4	11.3	13.17	1.00	0.45	1.86
31	150	8974	17.7	7.3	11.5	13.17	1.00	0.46	1.64
32	150	8980	17.7	7.1	11.6	13.17	1.00	0.47	1.81
33	150	8986	17.7	6.8	11.9	13.17	1.00	0.48	1.77
34	150	8992	17.7	6.5	12.3	13.17	1.00	0.49	1.72
35	150	8998	17.7	6.1	12.6	13.17	1.00	0.50	1.67
36	150	9004	17.7	5.8	12.9	13.17	1.00	0.52	1.63
37	150	9010	17.7	5.6	13.2	13.17	1.00	0.53	1.60
38	150	9016	17.7	5.4	13.4	13.17	1.00	0.54	1.57
39	150	9022	17.7	5.2	13.6	13.17	1.00	0.54	1.55
40	150	9028	17.7	5.0	13.8	13.17	1.00	0.55	1.53
41	150	9034	17.7	4.8	13.9	13.17	1.00	0.56	1.51



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 88+00 to 90+00
Critical Station = 90+00
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
25.0
20.0
15.0
10.0
5.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 88+00 to 90+00
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	50	ft
h_s =	17.90	ft
TOL =	766.7	ft elevation
Landside =	745.0	ft elevation
Bottom Blanket =	720	ft elevation
blanket =	25.0	ft
z, Landside Toe =	150	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	10.19	cfs

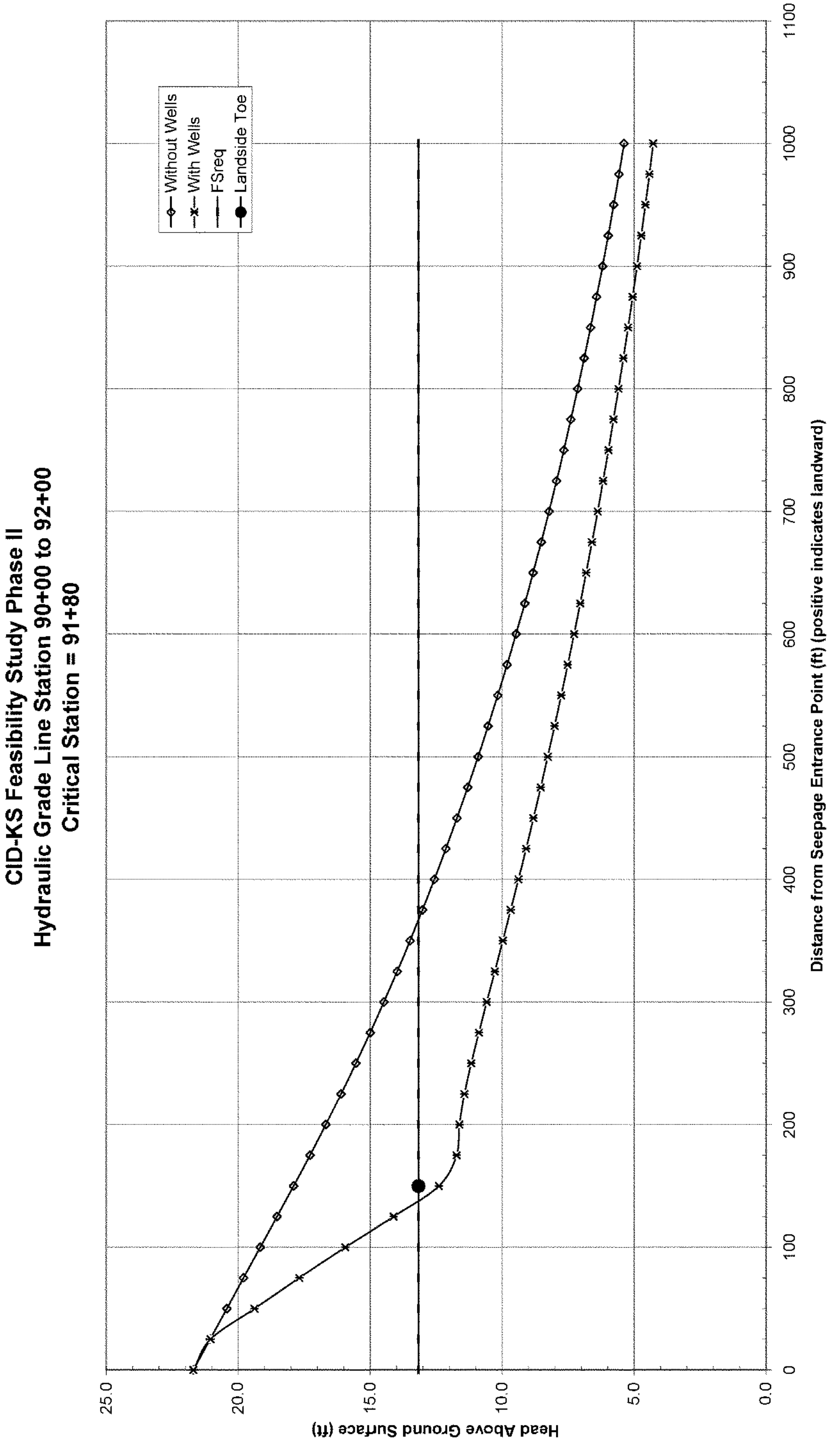
real well locations								image well locations		
well	x	y	discharge el	Q_w (cfs)	V_w (ft/s)	H_w (ft)		well	x'	y'
13	169	8776	745.0	0.87	0.28	1.00		13	-169	8776
14	162	8980	747.0	0.77	0.25	1.00		14	-162	8980
15	160	9200	748.0	0.81	0.26	1.00		15	-160	9200
16	159	9436	750.0	0.87	0.28	1.00		16	-159	9436
69	150	8760	742.0	1.02	0.33	1.00		69	-150	8760
70	150	8820	742.0	1.04	0.33	1.00		70	-150	8820
71	150	8950	745.0	0.89	0.28	1.00		71	-150	8950
72	150	9075	745.0	0.93	0.30	1.00		72	-150	9075
73	150	9150	745.0	0.94	0.30	1.00		73	-150	9150
				0.00	0.00	0.00		0	0	0
				0.00	0.00	0.00		0	0	0
				0.00	0.00	0.00		0	0	0
				0.00	0.00	0.00		0	0	0
				0.00	0.00	0.00		0	0	0
				0.00	0.00	0.00		0	0	0
				0.00	0.00	0.00		0	0	0
				0.00	0.00	0.00		0	0	0
				0.00	0.00	0.00		0	0	0
				0.00	0.00	0.00		0	0	0
				0.00	0.00	0.00		0	0	0
						1.00	**<---Input H_w AVG after any changes are made to well parameters**			

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

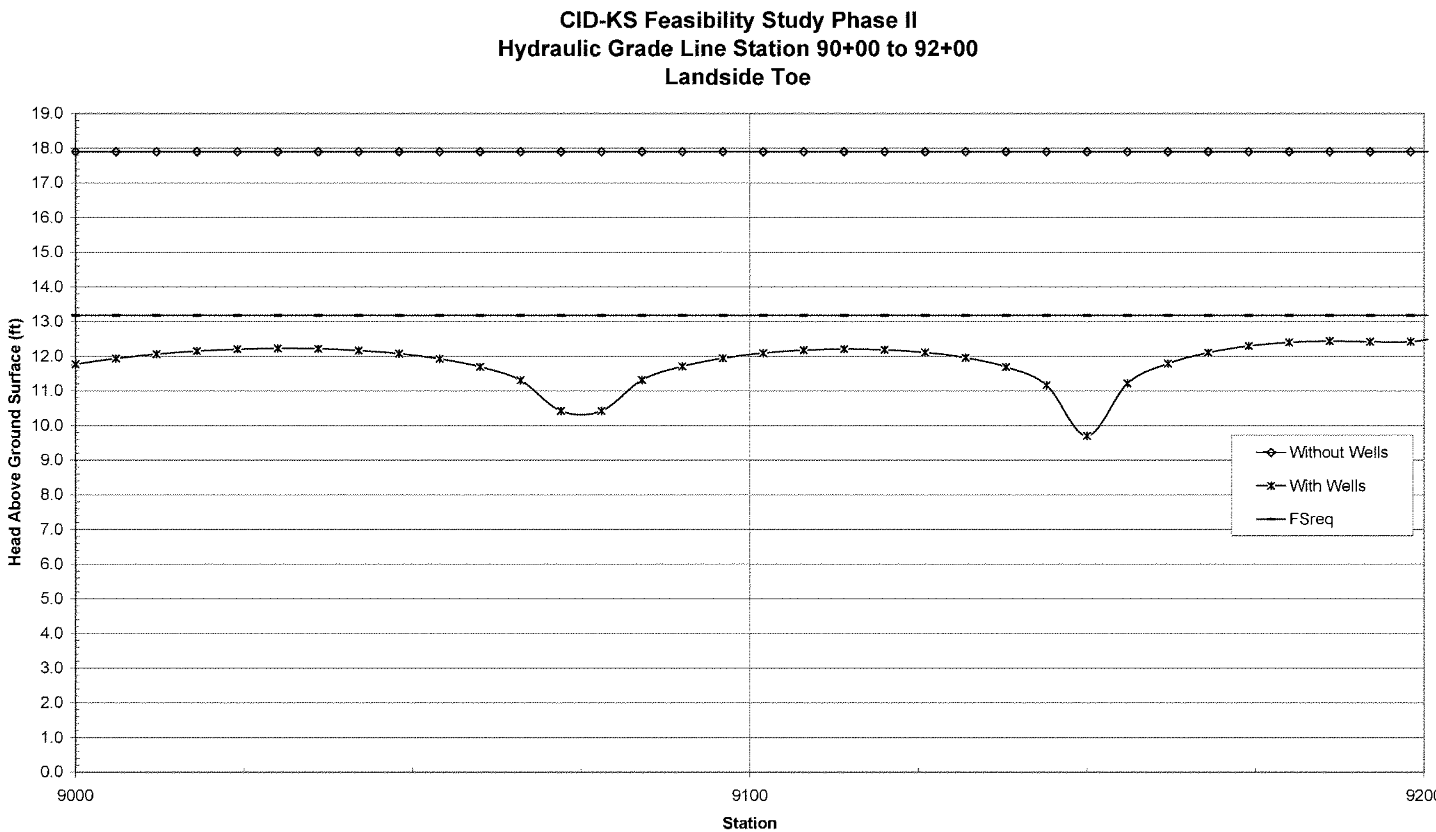
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	9180	21.7	0.0	21.7	13.17	1.00	0.87	0.97
2	25	9180	21.1	1.0	21.0	13.17	1.00	0.84	1.00
3	50	9180	20.4	2.1	19.4	13.17	1.00	0.78	1.09
4	75	9180	19.8	3.1	17.7	13.17	1.00	0.71	1.19
5	100	9180	19.2	4.2	15.9	13.17	1.00	0.64	1.32
6	125	9180	18.5	5.4	14.1	13.17	1.00	0.56	1.49
7	150	9180	17.9	6.5	12.4	13.17	1.00	0.50	1.70
8	175	9180	17.3	6.6	11.7	13.17	1.00	0.47	1.80
9	200	9180	16.7	6.1	11.6	13.17	1.00	0.46	1.81
10	225	9180	16.1	5.7	11.4	13.17	1.00	0.46	1.84
11	250	9180	15.5	5.4	11.2	13.17	1.00	0.45	1.89
12	275	9180	15.0	5.1	10.9	13.17	1.00	0.44	1.94
13	300	9180	14.5	4.9	10.6	13.17	1.00	0.42	1.99
14	325	9180	14.0	4.7	10.3	13.17	1.00	0.41	2.05
15	350	9180	13.5	4.5	10.0	13.17	1.00	0.40	2.11
16	375	9180	13.0	4.3	9.7	13.17	1.00	0.39	2.18
17	400	9180	12.6	4.2	9.4	13.17	1.00	0.38	2.25
18	425	9180	12.1	4.0	9.1	13.17	1.00	0.36	2.32
19	450	9180	11.7	3.9	8.8	13.17	1.00	0.35	2.39
20	475	9180	11.3	3.8	8.5	13.17	1.00	0.34	2.47
21	500	9180	10.9	3.6	8.3	13.17	1.00	0.33	2.55
22	525	9180	10.5	3.5	8.0	13.17	1.00	0.32	2.63
23	550	9180	10.2	3.4	7.8	13.17	1.00	0.31	2.72
24	575	9180	9.8	3.3	7.5	13.17	1.00	0.30	2.80
25	600	9180	9.5	3.2	7.3	13.17	1.00	0.29	2.90
26	625	9180	9.1	3.1	7.0	13.17	1.00	0.28	2.99
27	650	9180	8.8	3.0	6.8	13.17	1.00	0.27	3.09
28	675	9180	8.5	2.9	6.6	13.17	1.00	0.26	3.19
29	700	9180	8.2	2.8	6.4	13.17	1.00	0.26	3.30
30	725	9180	7.9	2.8	6.2	13.17	1.00	0.25	3.41
31	750	9180	7.7	2.7	6.0	13.17	1.00	0.24	3.53
32	775	9180	7.4	2.6	5.8	13.17	1.00	0.23	3.65
33	800	9180	7.1	2.5	5.6	13.17	1.00	0.22	3.77
34	825	9180	6.9	2.5	5.4	13.17	1.00	0.22	3.90
35	850	9180	6.7	2.4	5.2	13.17	1.00	0.21	4.03
36	875	9180	6.4	2.4	5.1	13.17	1.00	0.20	4.17
37	900	9180	6.2	2.3	4.9	13.17	1.00	0.20	4.31
38	925	9180	6.0	2.3	4.7	13.17	1.00	0.19	4.45
39	950	9180	5.8	2.2	4.6	13.17	1.00	0.18	4.61
40	975	9180	5.6	2.2	4.4	13.17	1.00	0.18	4.76
41	1000	9180	5.4	2.1	4.3	13.17	1.00	0.17	4.93

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	150	8994	17.9	7.3	11.6	13.17	1.00	0.46	1.82
2	150	9000	17.9	7.1	11.8	13.17	1.00	0.47	1.79
3	150	9006	17.9	7.0	11.9	13.17	1.00	0.48	1.77
4	150	9012	17.9	6.8	12.1	13.17	1.00	0.48	1.75
5	150	9018	17.9	6.8	12.1	13.17	1.00	0.49	1.73
6	150	9024	17.9	6.7	12.2	13.17	1.00	0.49	1.73
7	150	9030	17.9	6.7	12.2	13.17	1.00	0.49	1.72
8	150	9036	17.9	6.7	12.2	13.17	1.00	0.49	1.73
9	150	9042	17.9	6.7	12.2	13.17	1.00	0.49	1.73
10	150	9048	17.9	6.8	12.1	13.17	1.00	0.48	1.75
11	150	9054	17.9	7.0	11.9	13.17	1.00	0.48	1.77
12	150	9060	17.9	7.2	11.7	13.17	1.00	0.47	1.80
13	150	9066	17.9	7.6	11.3	13.17	1.00	0.45	1.86
14	150	9072	17.9	8.5	10.4	13.17	1.00	0.42	2.02
15	150	9078	17.9	8.5	10.4	13.17	1.00	0.42	2.02
16	150	9084	17.9	7.6	11.3	13.17	1.00	0.45	1.86
17	150	9090	17.9	7.2	11.7	13.17	1.00	0.47	1.80
18	150	9056	17.9	7.0	11.9	13.17	1.00	0.48	1.76
19	150	9102	17.9	6.8	12.1	13.17	1.00	0.48	1.74
20	150	9108	17.9	6.7	12.2	13.17	1.00	0.49	1.73
21	150	9114	17.9	6.7	12.2	13.17	1.00	0.49	1.73
22	150	9120	17.9	6.7	12.2	13.17	1.00	0.49	1.73
23	150	9126	17.9	6.8	12.1	13.17	1.00	0.48	1.74
24	150	9132	17.9	6.9	12.0	13.17	1.00	0.48	1.76
25	150	9138	17.9	7.2	11.7	13.17	1.00	0.47	1.80
26	150	9144	17.9	7.7	11.2	13.17	1.00	0.45	1.89
27	150	9150	17.9	9.2	9.7	13.17	1.00	0.39	2.17
28	150	9156	17.9	7.7	11.2	13.17	1.00	0.45	1.88
29	150	9162	17.9	7.1	11.8	13.17	1.00	0.47	1.79
30	150	9168	17.9	6.8	12.1	13.17	1.00	0.48	1.74
31	150	9174	17.9	6.6	12.3	13.17	1.00	0.49	1.71
32	150	9180	17.9	6.5	12.4	13.17	1.00	0.50	1.70
33	150	9186	17.9	6.5	12.4	13.17	1.00	0.50	1.69
34	150	9192	17.9	6.5	12.4	13.17	1.00	0.50	1.70
35	150	9198	17.9	6.5	12.4	13.17	1.00	0.50	1.70
36	150	9204	17.9	6.3	12.6	13.17	1.00	0.50	1.67
37	150	9210	17.9	6.0	12.9	13.17	1.00	0.52	1.63
38	150	9216	17.9	5.6	13.3	13.17	1.00	0.53	1.59
39	150	9222	17.9	5.3	13.6	13.17	1.00	0.54	1.55
40	150	9228	17.9	5.1	13.8	13.17	1.00	0.55	1.53
41	150	9234	17.9	4.9	14.0	13.17	1.00	0.56	1.50



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 90+00 to 92+00
Critical Station = 91+80
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
25.0
20.0
15.0
10.0
5.0
0.0
Distance from Seepage Entrance Point (ft) (positive indicates landward)
0
100
200
300
400
500
600
700
800
900
1000
1100


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 90+00 to 92+00
Landside Toe
Head Above Ground Surface (ft)
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
9000
9100
9200
Station
Without Wells
With Wells
FSreq

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	50	ft
h_0 =	15.44	ft
TOL =	796.9	ft elevation
Landside =	747.0	ft elevation
Bottom Blanket =	720	ft elevation
blanket =	27.0	ft
z, Landside Toe =	150	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	5.83	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
14	162	8980	747.0	1.06	0.34	1.00
15	160	9200	748.0	0.89	0.28	1.00
16	159	9436	750.0	0.86	0.27	1.00
17	160	9700	750.9	0.84	0.27	1.00
73	150	9150	745.0	1.09	0.35	1.01
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

1.00 <---**Input H_w AVG after any changes are made to well parameters**

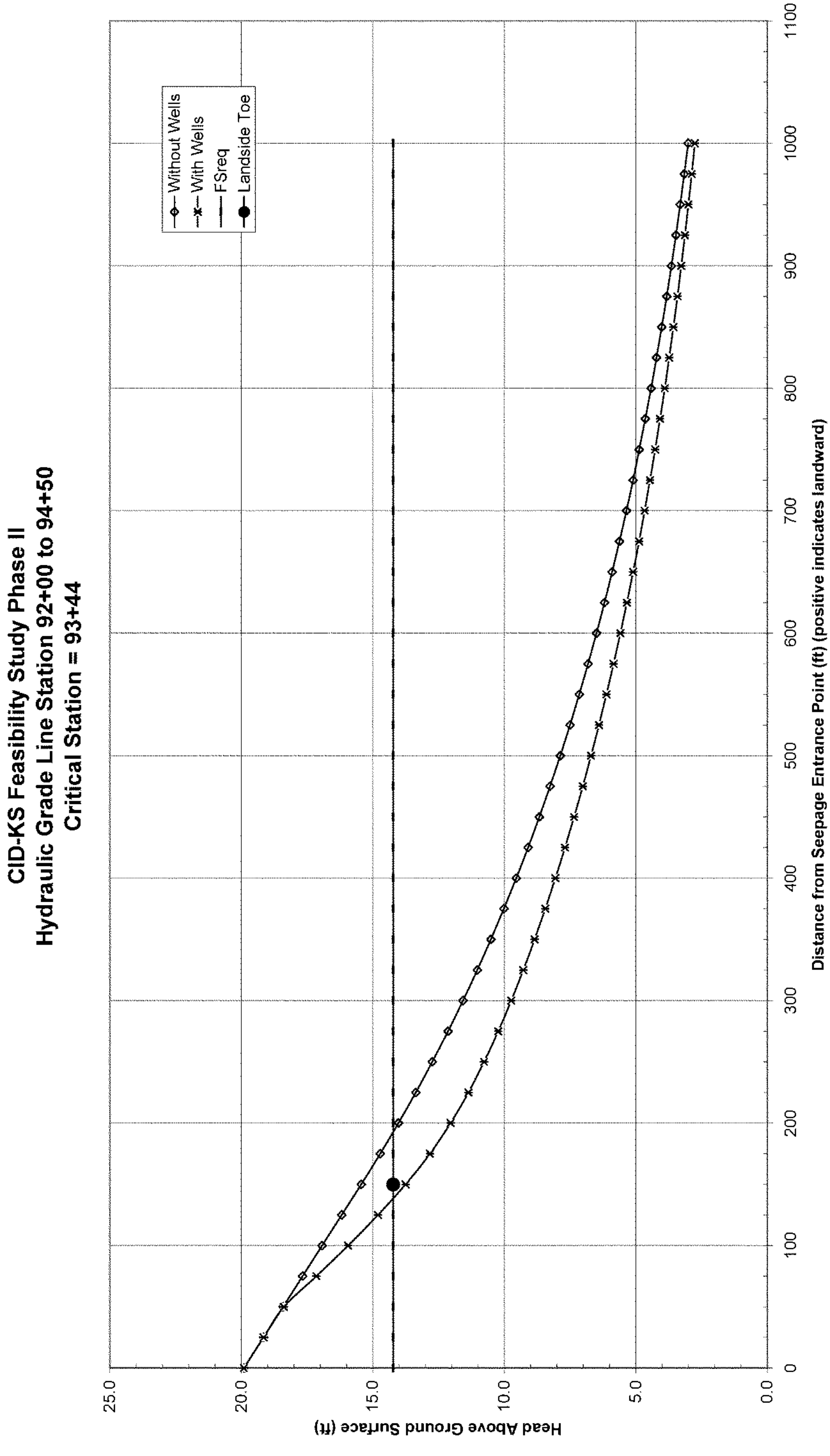
image well locations		
well	x'	y'
14	-162	8980
15	-160	9200
16	-159	9436
17	-160	9700
73	-150	9150
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

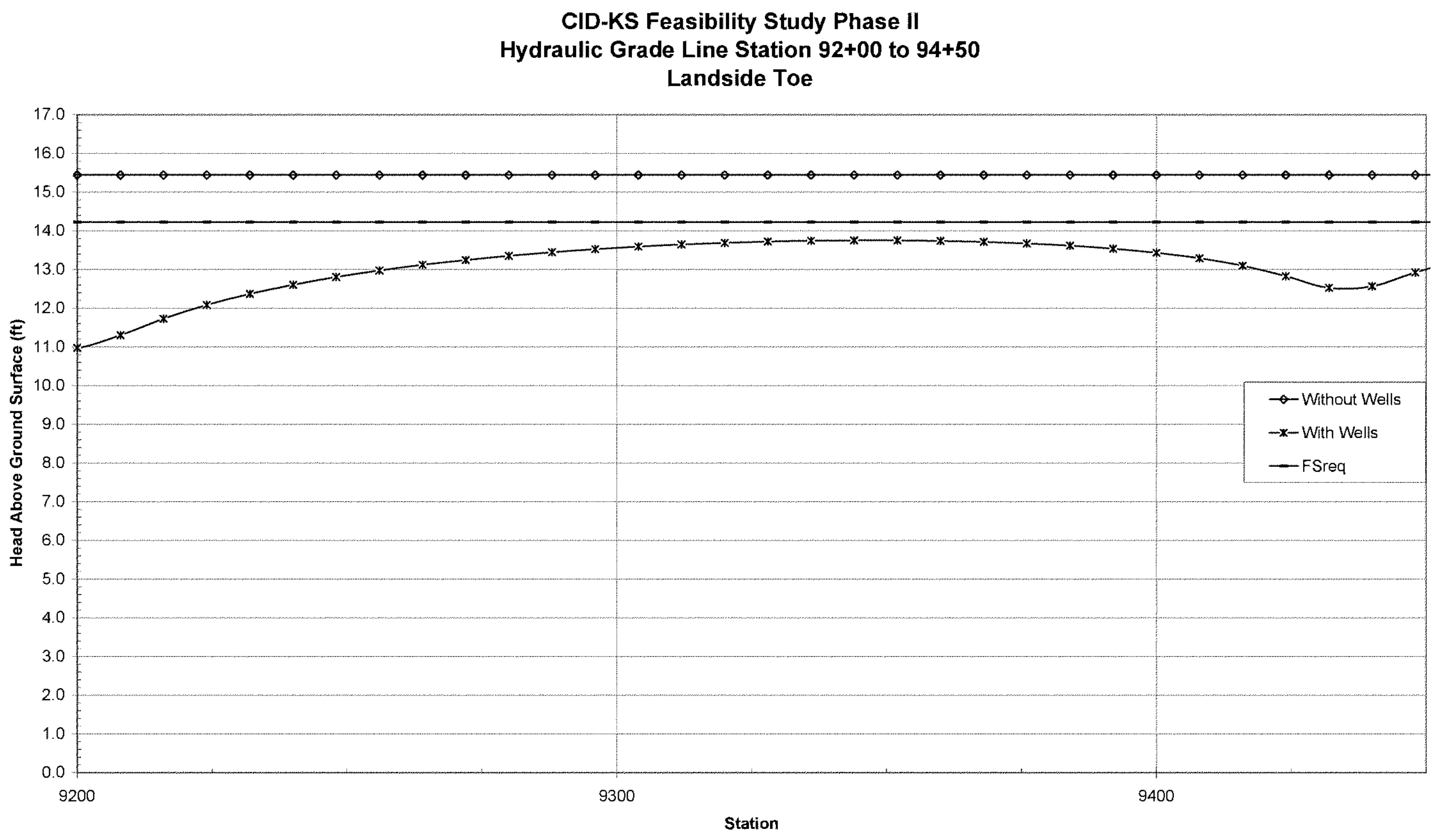
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	9344	19.9	0.0	19.9	14.22	1.00	0.74	1.14
2	25	9344	19.2	0.5	19.2	14.22	1.00	0.71	1.19
3	50	9344	18.4	1.0	18.4	14.22	1.00	0.68	1.24
4	75	9344	17.7	1.5	17.1	14.22	1.00	0.63	1.33
5	100	9344	16.9	2.0	15.9	14.22	1.00	0.59	1.43
6	125	9344	16.2	2.4	14.8	14.22	1.00	0.55	1.54
7	150	9344	15.4	2.7	13.8	14.22	1.00	0.51	1.65
8	175	9344	14.7	2.9	12.8	14.22	1.00	0.48	1.77
9	200	9344	14.0	3.0	12.0	14.22	1.00	0.45	1.89
10	225	9344	13.4	3.0	11.4	14.22	1.00	0.42	2.00
11	250	9344	12.7	3.0	10.8	14.22	1.00	0.40	2.11
12	275	9344	12.1	2.9	10.2	14.22	1.00	0.38	2.22
13	300	9344	11.5	2.8	9.7	14.22	1.00	0.36	2.34
14	325	9344	11.0	2.7	9.3	14.22	1.00	0.34	2.45
15	350	9344	10.5	2.7	8.8	14.22	1.00	0.33	2.57
16	375	9344	10.0	2.6	8.4	14.22	1.00	0.31	2.70
17	400	9344	9.5	2.5	8.1	14.22	1.00	0.30	2.82
18	425	9344	9.1	2.4	7.7	14.22	1.00	0.28	2.96
19	450	9344	8.7	2.3	7.3	14.22	1.00	0.27	3.10
20	475	9344	8.3	2.2	7.0	14.22	1.00	0.26	3.24
21	500	9344	7.9	2.2	6.7	14.22	1.00	0.25	3.40
22	525	9344	7.5	2.1	6.4	14.22	1.00	0.24	3.55
23	550	9344	7.2	2.0	6.1	14.22	1.00	0.23	3.72
24	575	9344	6.8	2.0	5.8	14.22	1.00	0.22	3.89
25	600	9344	6.5	1.9	5.6	14.22	1.00	0.21	4.07
26	625	9344	6.2	1.9	5.3	14.22	1.00	0.20	4.26
27	650	9344	5.9	1.8	5.1	14.22	1.00	0.19	4.46
28	675	9344	5.6	1.7	4.9	14.22	1.00	0.18	4.67
29	700	9344	5.4	1.7	4.7	14.22	1.00	0.17	4.88
30	725	9344	5.1	1.6	4.5	14.22	1.00	0.17	5.11
31	750	9344	4.9	1.6	4.3	14.22	1.00	0.16	5.34
32	775	9344	4.6	1.6	4.1	14.22	1.00	0.15	5.58
33	800	9344	4.4	1.5	3.9	14.22	1.00	0.14	5.84
34	825	9344	4.2	1.5	3.7	14.22	1.00	0.14	6.10
35	850	9344	4.0	1.4	3.6	14.22	1.00	0.13	6.37
36	875	9344	3.8	1.4	3.4	14.22	1.00	0.13	6.66
37	900	9344	3.6	1.4	3.3	14.22	1.00	0.12	6.96
38	925	9344	3.5	1.3	3.1	14.22	1.00	0.12	7.26
39	950	9344	3.3	1.3	3.0	14.22	1.00	0.11	7.58
40	975	9344	3.2	1.3	2.9	14.22	1.00	0.11	7.91
41	1000	9344	3.0	1.3	2.8	14.22	1.00	0.10	8.26

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	150	9192	15.4	5.4	11.0	14.22	1.00	0.41	2.07
2	150	9200	15.4	5.5	11.0	14.22	1.00	0.41	2.07
3	150	9208	15.4	5.1	11.3	14.22	1.00	0.42	2.01
4	150	9216	15.4	4.7	11.7	14.22	1.00	0.43	1.94
5	150	9224	15.4	4.4	12.1	14.22	1.00	0.45	1.88
6	150	9232	15.4	4.1	12.4	14.22	1.00	0.46	1.84
7	150	9240	15.4	3.8	12.6	14.22	1.00	0.47	1.81
8	150	9248	15.4	3.6	12.8	14.22	1.00	0.47	1.78
9	150	9256	15.4	3.5	13.0	14.22	1.00	0.48	1.75
10	150	9264	15.4	3.3	13.1	14.22	1.00	0.49	1.73
11	150	9272	15.4	3.2	13.2	14.22	1.00	0.49	1.72
12	150	9280	15.4	3.1	13.4	14.22	1.00	0.49	1.70
13	150	9288	15.4	3.0	13.4	14.22	1.00	0.50	1.69
14	150	9296	15.4	2.9	13.5	14.22	1.00	0.50	1.68
15	150	9304	15.4	2.9	13.6	14.22	1.00	0.50	1.67
16	150	9312	15.4	2.8	13.6	14.22	1.00	0.51	1.67
17	150	9320	15.4	2.8	13.7	14.22	1.00	0.51	1.66
18	150	9328	15.4	2.7	13.7	14.22	1.00	0.51	1.66
19	150	9336	15.4	2.7	13.7	14.22	1.00	0.51	1.66
20	150	9344	15.4	2.7	13.8	14.22	1.00	0.51	1.65
21	150	9352	15.4	2.7	13.8	14.22	1.00	0.51	1.65
22	150	9360	15.4	2.7	13.7	14.22	1.00	0.51	1.66
23	150	9368	15.4	2.7	13.7	14.22	1.00	0.51	1.66
24	150	9376	15.4	2.8	13.7	14.22	1.00	0.51	1.66
25	150	9384	15.4	2.8	13.6	14.22	1.00	0.50	1.67
26	150	9392	15.4	2.9	13.5	14.22	1.00	0.50	1.68
27	150	9400	15.4	3.0	13.4	14.22	1.00	0.50	1.69
28	150	9408	15.4	3.2	13.3	14.22	1.00	0.49	1.71
29	150	9416	15.4	3.3	13.1	14.22	1.00	0.49	1.74
30	150	9424	15.4	3.6	12.8	14.22	1.00	0.48	1.77
31	150	9432	15.4	3.9	12.5	14.22	1.00	0.46	1.82
32	150	9440	15.4	3.9	12.6	14.22	1.00	0.47	1.81
33	150	9448	15.4	3.5	12.9	14.22	1.00	0.48	1.76
34	150	9456	15.4	3.2	13.2	14.22	1.00	0.49	1.72
35	150	9464	15.4	3.0	13.5	14.22	1.00	0.50	1.69
36	150	9472	15.4	2.8	13.7	14.22	1.00	0.51	1.67
37	150	9480	15.4	2.6	13.8	14.22	1.00	0.51	1.65
38	150	9488	15.4	2.5	13.9	14.22	1.00	0.52	1.63
39	150	9496	15.4	2.4	14.0	14.22	1.00	0.52	1.62
40	150	9504	15.4	2.3	14.1	14.22	1.00	0.52	1.61
41	150	9512	15.4	2.2	14.2	14.22	1.00	0.53	1.60


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 92+00 to 94+50
Critical Station = 93+44
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
25.0
20.0
15.0
10.0
5.0
0.0
Distance from Seepage Entrance Point (ft) (positive indicates landward)
0
100
200
300
400
500
600
700
800
900
1000
1100


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 92+00 to 94+50
Landside Toe
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
9200
9300
9400
Station
Without Wells
With Wells
FSreq

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	50	ft
h_o =	14.24	ft
TOL =	767	ft elevation
Landside =	750.0	ft elevation
Bottom Blanket =	720	ft elevation
blanket =	30.0	ft
z, Landside Toe =	150	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	5.36	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
15	160	9200	748.0	1.05	0.33	1.00
16	159	9436	750.0	1.00	0.32	1.00
17	160	9700	750.0	0.98	0.31	1.00
73	150	9150	745.0	1.27	0.40	1.01
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

<--Input H_w AVG after any changes are made to well parameters

image well locations		
well	x'	y'
15	-160	9200
16	-159	9436
17	-160	9700
73	-150	9150
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

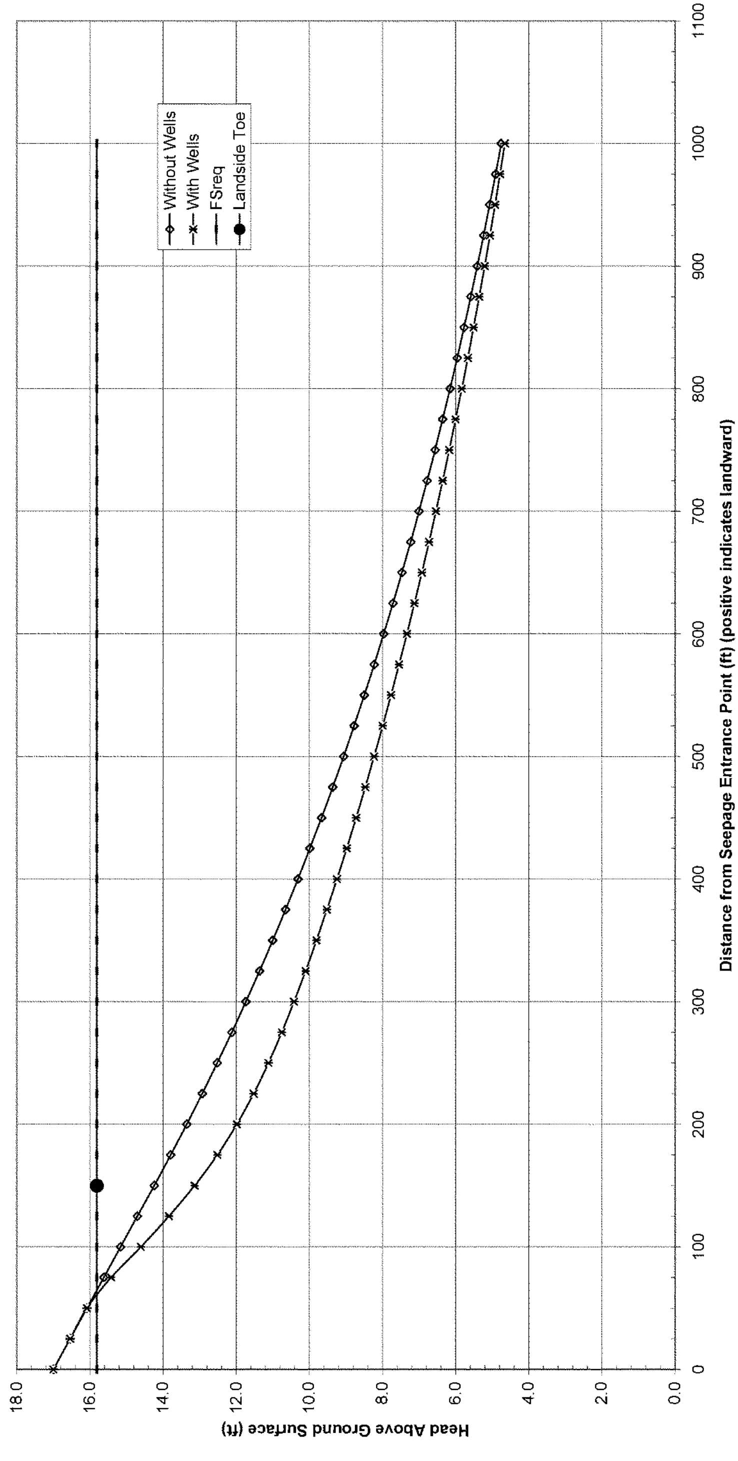
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	9570	17.0	0.0	17.0	15.81	1.00	0.57	1.49
2	25	9570	16.5	0.4	16.5	15.81	1.00	0.55	1.53
3	50	9570	16.1	0.8	16.1	15.81	1.00	0.54	1.57
4	75	9570	15.6	1.2	15.4	15.81	1.00	0.51	1.64
5	100	9570	15.2	1.6	14.6	15.81	1.00	0.49	1.73
6	125	9570	14.7	1.9	13.8	15.81	1.00	0.46	1.83
7	150	9570	14.2	2.1	13.1	15.81	1.00	0.44	1.92
8	175	9570	13.8	2.3	12.5	15.81	1.00	0.42	2.02
9	200	9570	13.4	2.4	12.0	15.81	1.00	0.40	2.11
10	225	9570	12.9	2.4	11.5	15.81	1.00	0.38	2.20
11	250	9570	12.5	2.4	11.1	15.81	1.00	0.37	2.27
12	275	9570	12.1	2.4	10.8	15.81	1.00	0.36	2.35
13	300	9570	11.7	2.3	10.4	15.81	1.00	0.35	2.43
14	325	9570	11.4	2.3	10.1	15.81	1.00	0.34	2.50
15	350	9570	11.0	2.2	9.8	15.81	1.00	0.33	2.58
16	375	9570	10.7	2.1	9.5	15.81	1.00	0.32	2.66
17	400	9570	10.3	2.1	9.2	15.81	1.00	0.31	2.74
18	425	9570	10.0	2.0	9.0	15.81	1.00	0.30	2.82
19	450	9570	9.7	1.9	8.7	15.81	1.00	0.29	2.90
20	475	9570	9.4	1.9	8.5	15.81	1.00	0.28	2.99
21	500	9570	9.1	1.8	8.2	15.81	1.00	0.27	3.07
22	525	9570	8.8	1.8	8.0	15.81	1.00	0.27	3.16
23	550	9570	8.5	1.7	7.8	15.81	1.00	0.26	3.26
24	575	9570	8.2	1.7	7.5	15.81	1.00	0.25	3.35
25	600	9570	8.0	1.6	7.3	15.81	1.00	0.24	3.45
26	625	9570	7.7	1.6	7.1	15.81	1.00	0.24	3.55
27	650	9570	7.5	1.5	6.9	15.81	1.00	0.23	3.65
28	675	9570	7.2	1.5	6.7	15.81	1.00	0.22	3.76
29	700	9570	7.0	1.5	6.5	15.81	1.00	0.22	3.87
30	725	9570	6.8	1.4	6.4	15.81	1.00	0.21	3.98
31	750	9570	6.6	1.4	6.2	15.81	1.00	0.21	4.10
32	775	9570	6.4	1.4	6.0	15.81	1.00	0.20	4.21
33	800	9570	6.2	1.3	5.8	15.81	1.00	0.19	4.34
34	825	9570	6.0	1.3	5.7	15.81	1.00	0.19	4.46
35	850	9570	5.8	1.3	5.5	15.81	1.00	0.18	4.59
36	875	9570	5.6	1.2	5.4	15.81	1.00	0.18	4.72
37	900	9570	5.4	1.2	5.2	15.81	1.00	0.17	4.86
38	925	9570	5.2	1.2	5.1	15.81	1.00	0.17	5.00
39	950	9570	5.1	1.1	4.9	15.81	1.00	0.16	5.14
40	975	9570	4.9	1.1	4.8	15.81	1.00	0.16	5.28
41	1000	9570	4.8	1.1	4.7	15.81	1.00	0.16	5.43

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	150	9442	14.2	4.2	11.0	15.81	1.00	0.37	2.29
2	150	9450	14.2	3.8	11.4	15.81	1.00	0.38	2.21
3	150	9458	14.2	3.5	11.8	15.81	1.00	0.39	2.15
4	150	9466	14.2	3.2	12.0	15.81	1.00	0.40	2.10
5	150	9474	14.2	3.0	12.2	15.81	1.00	0.41	2.07
6	150	9482	14.2	2.8	12.4	15.81	1.00	0.41	2.04
7	150	9490	14.2	2.7	12.5	15.81	1.00	0.42	2.02
8	150	9498	14.2	2.6	12.6	15.81	1.00	0.42	2.00
9	150	9506	14.2	2.5	12.7	15.81	1.00	0.42	1.98
10	150	9514	14.2	2.4	12.8	15.81	1.00	0.43	1.97
11	150	9522	14.2	2.3	12.9	15.81	1.00	0.43	1.96
12	150	9530	14.2	2.3	13.0	15.81	1.00	0.43	1.95
13	150	9538	14.2	2.2	13.0	15.81	1.00	0.43	1.94
14	150	9546	14.2	2.2	13.1	15.81	1.00	0.44	1.94
15	150	9554	14.2	2.1	13.1	15.81	1.00	0.44	1.93
16	150	9562	14.2	2.1	13.1	15.81	1.00	0.44	1.93
17	150	9570	14.2	2.1	13.1	15.81	1.00	0.44	1.92
18	150	9578	14.2	2.1	13.1	15.81	1.00	0.44	1.92
19	150	9586	14.2	2.1	13.2	15.81	1.00	0.44	1.92
20	150	9594	14.2	2.1	13.1	15.81	1.00	0.44	1.92
21	150	9602	14.2	2.1	13.1	15.81	1.00	0.44	1.93
22	150	9610	14.2	2.1	13.1	15.81	1.00	0.44	1.93
23	150	9618	14.2	2.2	13.1	15.81	1.00	0.44	1.93
24	150	9626	14.2	2.2	13.0	15.81	1.00	0.43	1.94
25	150	9634	14.2	2.3	13.0	15.81	1.00	0.43	1.95
26	150	9642	14.2	2.3	12.9	15.81	1.00	0.43	1.96
27	150	9650	14.2	2.4	12.8	15.81	1.00	0.43	1.97
28	150	9658	14.2	2.5	12.7	15.81	1.00	0.42	1.99
29	150	9666	14.2	2.7	12.6	15.81	1.00	0.42	2.01
30	150	9674	14.2	2.8	12.4	15.81	1.00	0.41	2.04
31	150	9682	14.2	3.1	12.2	15.81	1.00	0.41	2.08
32	150	9690	14.2	3.4	11.9	15.81	1.00	0.40	2.13
33	150	9698	14.2	3.6	11.6	15.81	1.00	0.39	2.18
34	150	9706	14.2	3.5	11.7	15.81	1.00	0.39	2.16
35	150	9714	14.2	3.2	12.1	15.81	1.00	0.40	2.09
36	150	9722	14.2	2.8	12.4	15.81	1.00	0.41	2.04
37	150	9730	14.2	2.6	12.7	15.81	1.00	0.42	2.00
38	150	9738	14.2	2.4	12.9	15.81	1.00	0.43	1.97
39	150	9746	14.2	2.2	13.0	15.81	1.00	0.43	1.94
40	150	9754	14.2	2.1	13.2	15.81	1.00	0.44	1.92
41	150	9762	14.2	1.9	13.3	15.81	1.00	0.44	1.90

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 94+50 to 97+00
Critical Station = 95+70



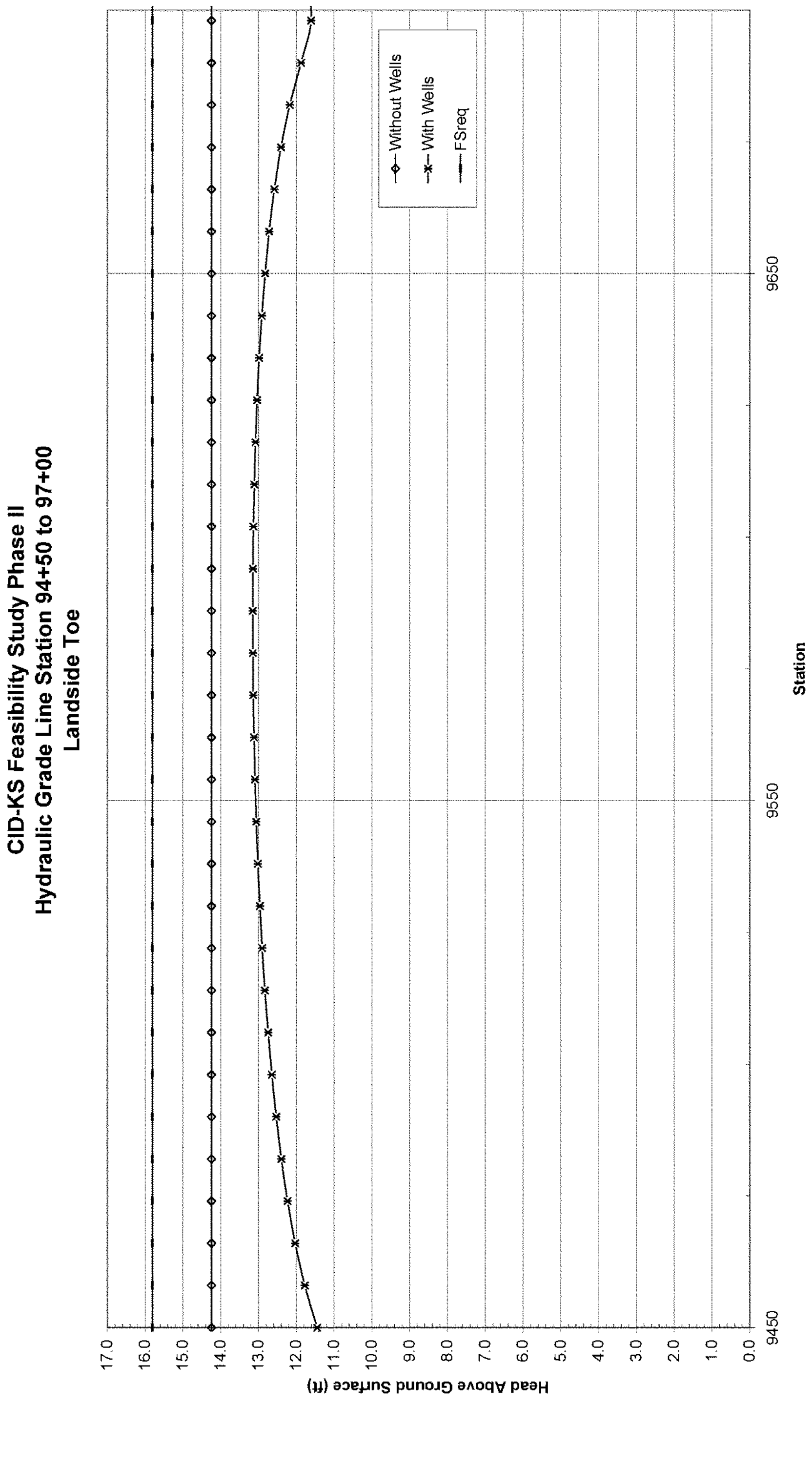

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 94+50 to 97+00
Landside Toe
Head Above Ground Surface (ft)
Station
9450
9550
9650
Without Wells
With Wells
FSreq

EXHIBIT A-4.18

Relief Well Design – Station 107+00 to 116+00

Project Name
UNDERSEEPAGE ANALYSIS WITHOUT WELLS
Levee Foundation Information - Case Analyzed

Blanket Unit Weight = 115.0 pcf (saturated)

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Blanket Elevation (msl)	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio River Side k_f/k_{br}	Permeability Ratio Land Side k_f/k_{bl}	Impervious Blanket Thickness (ft) River Side z_{br}	Impervious Blanket Thickness (ft) Levee z_{bb}	Impervious Blanket Thickness (ft) Land Side z_{bl}	Pervious Blanket Thickness (ft) d	Seepage Length (ft) River Side L_1	Seepage Length (ft) Levee L_2	Seepage Length (ft) Land Side L_3	Factor c_r	Factor c_l	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Factor of Safety for Piping (i_c/i_o)	Remarks
107+00 to 112+00	768.0	740	750	720	720	745	670	18.0	300	300	20	22.5	25	50	0	70	1000	0.001826	0.001633	0	612	16.15	0.65	0.84	1.30	**low area landward of levee.** fill effects
112+00 to 116+00	768.4	740	753	720	720	740	670	15.4	300	300	20	20	20	50	0	70	1000	0.001826	0.001826	0	546	13.65	0.68	0.84	1.23	fill effects
116+00 to 119+00	768.6	740	757	720	720	747	970	11.6	300	300	20	23.5	27	50	0	70	1000	0.001826	0.001571	0	636	10.45	0.39	0.84	2.16	fill effects

N500+3 Relief Well System Summary Station 107+00 to 116+00

Well	Distance From Seepage Entrance (ft)	Station	Discharge Elevation (ft)	0.8*Qw (cfs)	Qw (cfs)
1	70	106+80	757	0.54	0.68
2	70	107+00	757	0.51	0.64
3	70	107+30	757	0.50	0.62
4	70	107+60	757	0.50	0.62
5	70	108+00	757	0.50	0.63
6	70	108+40	757	0.50	0.63
7	70	108+80	757	0.50	0.63
8	70	109+20	757	0.50	0.63
9	70	109+60	757	0.50	0.63
10	70	110+00	757	0.64	0.80
11	70	110+40	757	0.58	0.72
12	70	110+80	757	0.55	0.68
13	70	111+20	757	0.52	0.65
14	70	111+50	757	0.50	0.63
15	70	111+80	757	0.49	0.62
16	70	112+10	757	0.52	0.65
17	70	112+40	757	0.57	0.71
18	70	112+85	757	0.53	0.66
19	70	113+30	757	0.53	0.66
20	70	113+70	757	0.53	0.66
21	70	114+10	757	0.53	0.66
22	70	114+50	757	0.53	0.66
23	70	114+90	757	0.53	0.66
24	70	115+30	757	0.53	0.66
25	70	115+60	757	0.53	0.66
26	70	115+90	757	0.55	0.69
27	70	116+20	757	0.60	0.75
			Total	**14.32**	**17.90**

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	50	ft
h_0 =	16.15	ft
TOL =	768	ft elevation
Landside =	750.0	ft elevation
Bottom Blanket =	720	ft elevation
blanket =	25.0	ft
z, Landside Toe =	70	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	10.78	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
1	70	10680	757.0	0.54	0.17	1.00
2	70	10700	757.0	0.51	0.16	1.00
3	70	10730	757.0	0.50	0.16	1.00
4	70	10760	757.0	0.50	0.16	1.00
5	70	10800	757.0	0.50	0.16	1.00
6	70	10840	757.0	0.50	0.16	1.00
7	70	10880	757.0	0.50	0.16	1.00
8	70	10920	757.0	0.50	0.16	1.00
9	70	10960	757.0	0.50	0.16	1.00
10	70	11000	757.0	0.50	0.16	1.00
11	70	11040	757.0	0.50	0.16	1.00
12	70	11080	757.0	0.50	0.16	1.00
13	70	11120	757.0	0.49	0.16	1.00
14	70	11150	757.0	0.49	0.15	1.00
15	70	11180	757.0	0.50	0.16	1.00
16	70	11210	757.0	0.52	0.16	1.00
17	70	11240	757.0	0.57	0.18	1.00
18				0.00	0.00	0.00
19				0.00	0.00	0.00
20				0.00	0.00	0.00
						1.00

1.00 <---Input H_w AVG after any changes are made to well parameters

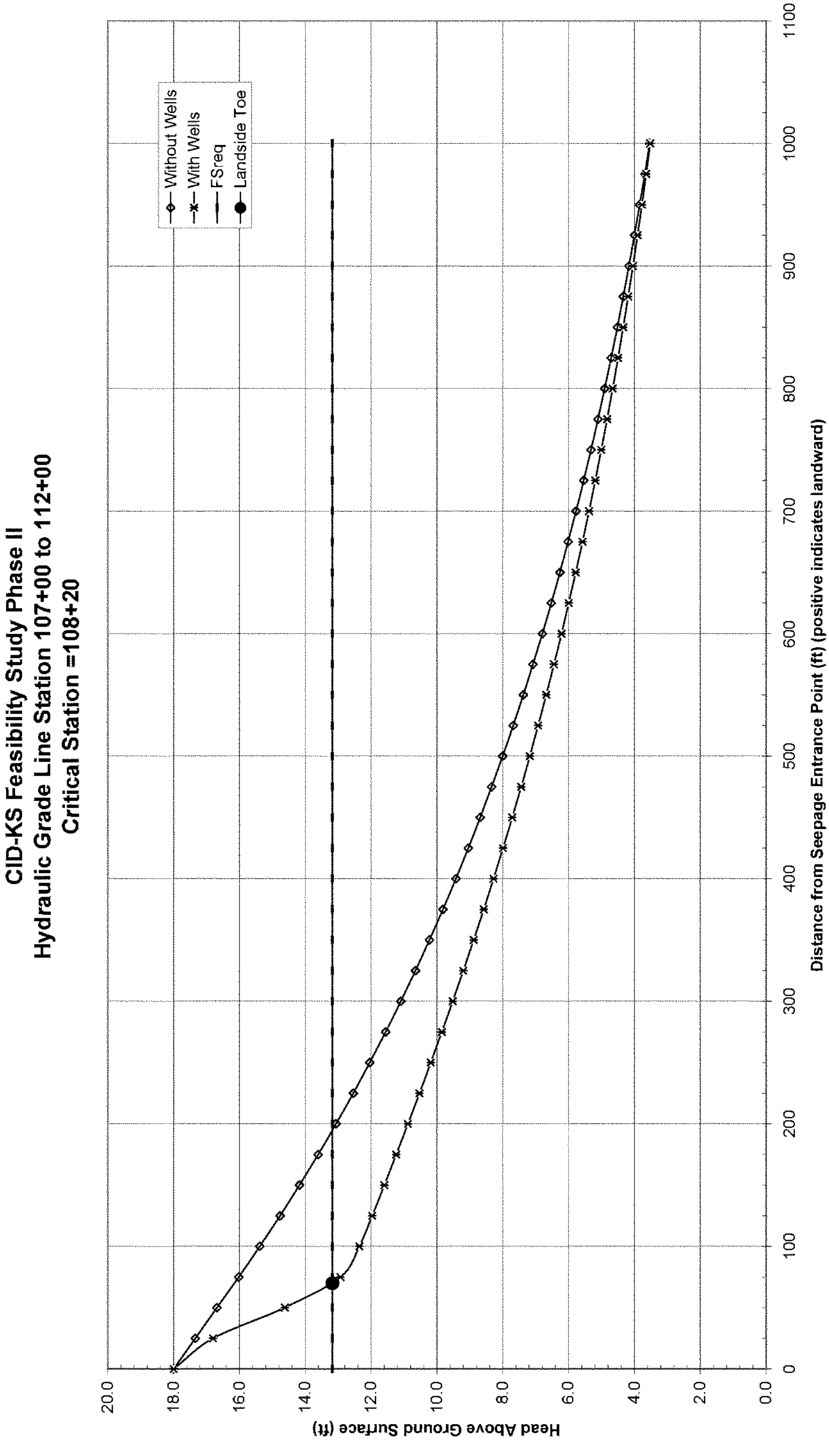
image well locations		
well	x'	y'
1	-70	10680
2	-70	10700
3	-70	10730
4	-70	10760
5	-70	10800
6	-70	10840
7	-70	10880
8	-70	10920
9	-70	10960
10	-70	11000
11	-70	11040
12	-70	11080
13	-70	11120
14	-70	11150
15	-70	11180
16	-70	11210
17	-70	11240
18	0	0
19	0	0
20	0	0

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

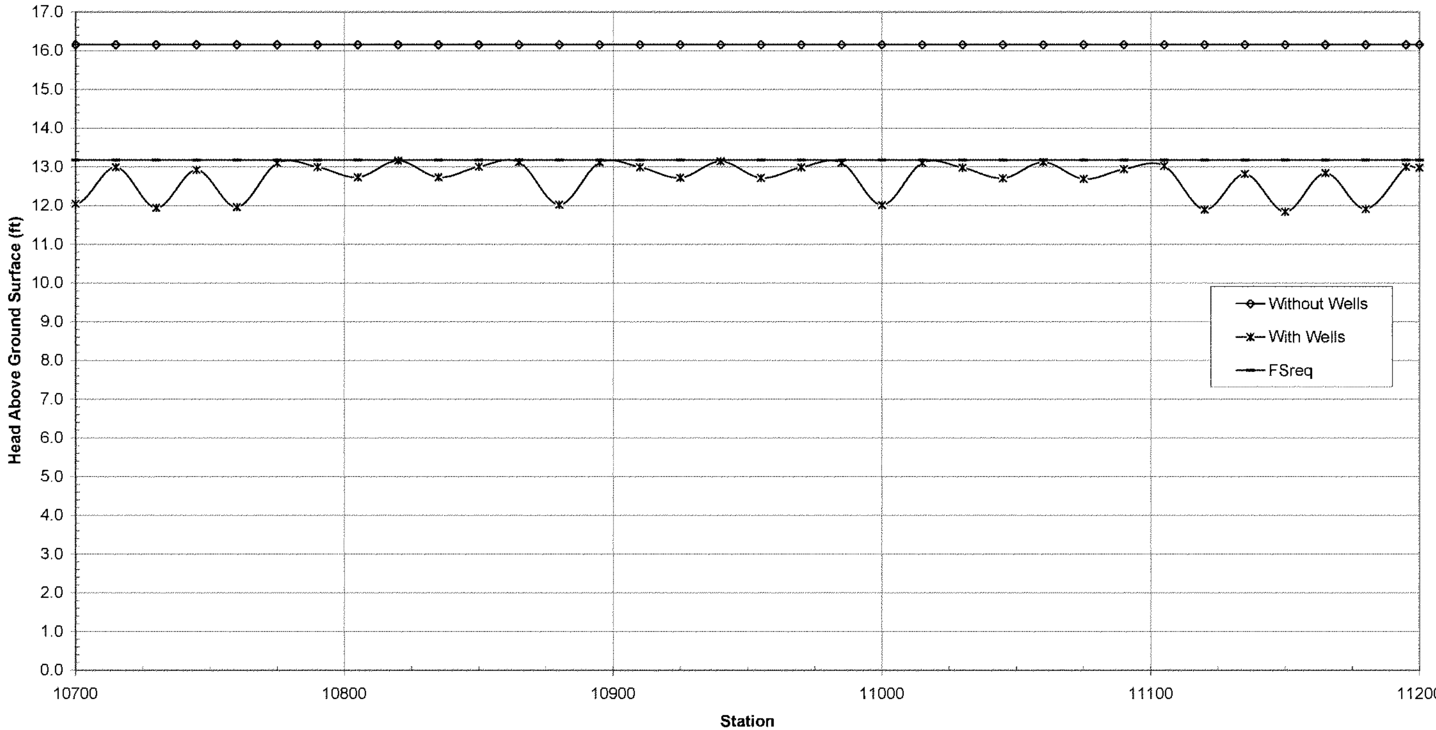
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	10820	18.0	0.0	18.0	13.17	1.00	0.72	1.17
2	25	10820	17.3	1.5	16.8	13.17	1.00	0.67	1.25
3	50	10820	16.7	3.1	14.6	13.17	1.00	0.58	1.44
4	75	10820	16.0	4.1	12.9	13.17	1.00	0.52	1.63
5	100	10820	15.4	4.0	12.4	13.17	1.00	0.49	1.71
6	125	10820	14.8	3.8	12.0	13.17	1.00	0.48	1.76
7	150	10820	14.2	3.6	11.6	13.17	1.00	0.46	1.82
8	175	10820	13.6	3.4	11.2	13.17	1.00	0.45	1.88
9	200	10820	13.1	3.2	10.9	13.17	1.00	0.44	1.94
10	225	10820	12.5	3.0	10.5	13.17	1.00	0.42	2.00
11	250	10820	12.0	2.9	10.2	13.17	1.00	0.41	2.07
12	275	10820	11.6	2.7	9.9	13.17	1.00	0.39	2.14
13	300	10820	11.1	2.6	9.5	13.17	1.00	0.38	2.21
14	325	10820	10.7	2.5	9.2	13.17	1.00	0.37	2.29
15	350	10820	10.2	2.3	8.9	13.17	1.00	0.36	2.37
16	375	10820	9.8	2.2	8.6	13.17	1.00	0.34	2.46
17	400	10820	9.4	2.1	8.3	13.17	1.00	0.33	2.54
18	425	10820	9.0	2.1	8.0	13.17	1.00	0.32	2.64
19	450	10820	8.7	2.0	7.7	13.17	1.00	0.31	2.73
20	475	10820	8.3	1.9	7.4	13.17	1.00	0.30	2.83
21	500	10820	8.0	1.8	7.2	13.17	1.00	0.29	2.94
22	525	10820	7.7	1.8	6.9	13.17	1.00	0.28	3.04
23	550	10820	7.4	1.7	6.7	13.17	1.00	0.27	3.15
24	575	10820	7.1	1.6	6.4	13.17	1.00	0.26	3.27
25	600	10820	6.8	1.6	6.2	13.17	1.00	0.25	3.39
26	625	10820	6.5	1.5	6.0	13.17	1.00	0.24	3.52
27	650	10820	6.3	1.5	5.8	13.17	1.00	0.23	3.64
28	675	10820	6.0	1.4	5.6	13.17	1.00	0.22	3.78
29	700	10820	5.8	1.4	5.4	13.17	1.00	0.22	3.92
30	725	10820	5.5	1.4	5.2	13.17	1.00	0.21	4.06
31	750	10820	5.3	1.3	5.0	13.17	1.00	0.20	4.21
32	775	10820	5.1	1.3	4.8	13.17	1.00	0.19	4.36
33	800	10820	4.9	1.2	4.7	13.17	1.00	0.19	4.52
34	825	10820	4.7	1.2	4.5	13.17	1.00	0.18	4.68
35	850	10820	4.5	1.2	4.3	13.17	1.00	0.17	4.85
36	875	10820	4.3	1.1	4.2	13.17	1.00	0.17	5.03
37	900	10820	4.2	1.1	4.0	13.17	1.00	0.16	5.21
38	925	10820	4.0	1.1	3.9	13.17	1.00	0.16	5.40
39	950	10820	3.8	1.1	3.8	13.17	1.00	0.15	5.59
40	975	10820	3.7	1.0	3.6	13.17	1.00	0.15	5.78
41	1000	10820	3.5	1.0	3.5	13.17	1.00	0.14	5.99

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	70	10685	16.2	4.2	13.0	13.17	1.00	0.52	1.63
2	70	10700	16.2	5.1	12.0	13.17	1.00	0.48	1.75
3	70	10715	16.2	4.2	13.0	13.17	1.00	0.52	1.62
4	70	10730	16.2	5.2	11.9	13.17	1.00	0.48	1.76
5	70	10745	16.2	4.2	12.9	13.17	1.00	0.52	1.63
6	70	10760	16.2	5.2	12.0	13.17	1.00	0.48	1.76
7	70	10775	16.2	4.1	13.1	13.17	1.00	0.52	1.61
8	70	10790	16.2	4.2	13.0	13.17	1.00	0.52	1.62
9	70	10805	16.2	4.4	12.7	13.17	1.00	0.51	1.66
10	70	10820	16.2	4.0	13.2	13.17	1.00	0.53	1.60
11	70	10835	16.2	4.4	12.7	13.17	1.00	0.51	1.66
12	70	10850	16.2	4.1	13.0	13.17	1.00	0.52	1.62
13	70	10865	16.2	4.0	13.1	13.17	1.00	0.52	1.61
14	70	10880	16.2	5.1	12.0	13.17	1.00	0.48	1.75
15	70	10895	16.2	4.0	13.1	13.17	1.00	0.52	1.61
16	70	10910	16.2	4.2	13.0	13.17	1.00	0.52	1.62
17	70	10925	16.2	4.4	12.7	13.17	1.00	0.51	1.66
18	70	10940	16.2	4.0	13.1	13.17	1.00	0.53	1.60
19	70	10955	16.2	4.4	12.7	13.17	1.00	0.51	1.66
20	70	10970	16.2	4.2	13.0	13.17	1.00	0.52	1.62
21	70	10985	16.2	4.1	13.1	13.17	1.00	0.52	1.61
22	70	11000	16.2	5.1	12.0	13.17	1.00	0.48	1.75
23	70	11015	16.2	4.1	13.1	13.17	1.00	0.52	1.61
24	70	11030	16.2	4.2	13.0	13.17	1.00	0.52	1.62
25	70	11045	16.2	4.4	12.7	13.17	1.00	0.51	1.66
26	70	11060	16.2	4.0	13.1	13.17	1.00	0.52	1.61
27	70	11075	16.2	4.5	12.7	13.17	1.00	0.51	1.66
28	70	11090	16.2	4.2	12.9	13.17	1.00	0.52	1.63
29	70	11105	16.2	4.1	13.0	13.17	1.00	0.52	1.62
30	70	11120	16.2	5.3	11.9	13.17	1.00	0.48	1.77
31	70	11135	16.2	4.3	12.8	13.17	1.00	0.51	1.64
32	70	11150	16.2	5.3	11.8	13.17	1.00	0.47	1.78
33	70	11165	16.2	4.3	12.8	13.17	1.00	0.51	1.64
34	70	11180	16.2	5.2	11.9	13.17	1.00	0.48	1.77
35	70	11195	16.2	4.2	13.0	13.17	1.00	0.52	1.62
36	70	11200	16.2	4.2	13.0	13.17	1.00	0.52	1.62
37	70	11225	16.2	3.8	13.4	13.17	1.00	0.53	1.58
38	70	11240	16.2	4.6	12.6	13.17	1.00	0.50	1.67
39	70	11255	16.2	2.8	14.3	13.17	1.00	0.57	1.47
40	70	11270	16.2	2.2	14.9	13.17	1.00	0.60	1.41
41	70	11285	16.2	1.8	15.3	13.17	1.00	0.61	1.37


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 107+00 to 112+00
Critical Station =108+20
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
0
100
200
300
400
500
600
700
800
900
1000
1100

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 107+00 to 112+00
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	50	ft
h_o =	13.65	ft
TOL =	768.4	ft elevation
Landside =	753.0	ft elevation
Bottom Blanket =	720	ft elevation
blanket =	20.0	ft
z, Landside Toe =	70	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	12.10	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
10	70	11000	757.0	0.64	0.20	1.00
11	70	11040	757.0	0.58	0.18	1.00
12	70	11080	757.0	0.55	0.17	1.00
13	70	11120	757.0	0.52	0.16	1.00
14	70	11150	757.0	0.50	0.16	1.00
15	70	11180	757.0	0.49	0.16	1.00
16	70	11210	757.0	0.50	0.16	1.00
17	70	11240	757.0	0.51	0.16	1.00
18	70	11285	757.0	0.53	0.17	1.00
19	70	11330	757.0	0.53	0.17	1.00
20	70	11370	757.0	0.53	0.17	1.00
21	70	11410	757.0	0.53	0.17	1.00
22	70	11450	757.0	0.53	0.17	1.00
23	70	11490	757.0	0.53	0.17	1.00
24	70	11530	757.0	0.53	0.17	1.00
25	70	11560	757.0	0.53	0.17	1.00
26	70	11590	757.0	0.55	0.18	1.00
27	70	11620	757.0	0.60	0.19	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

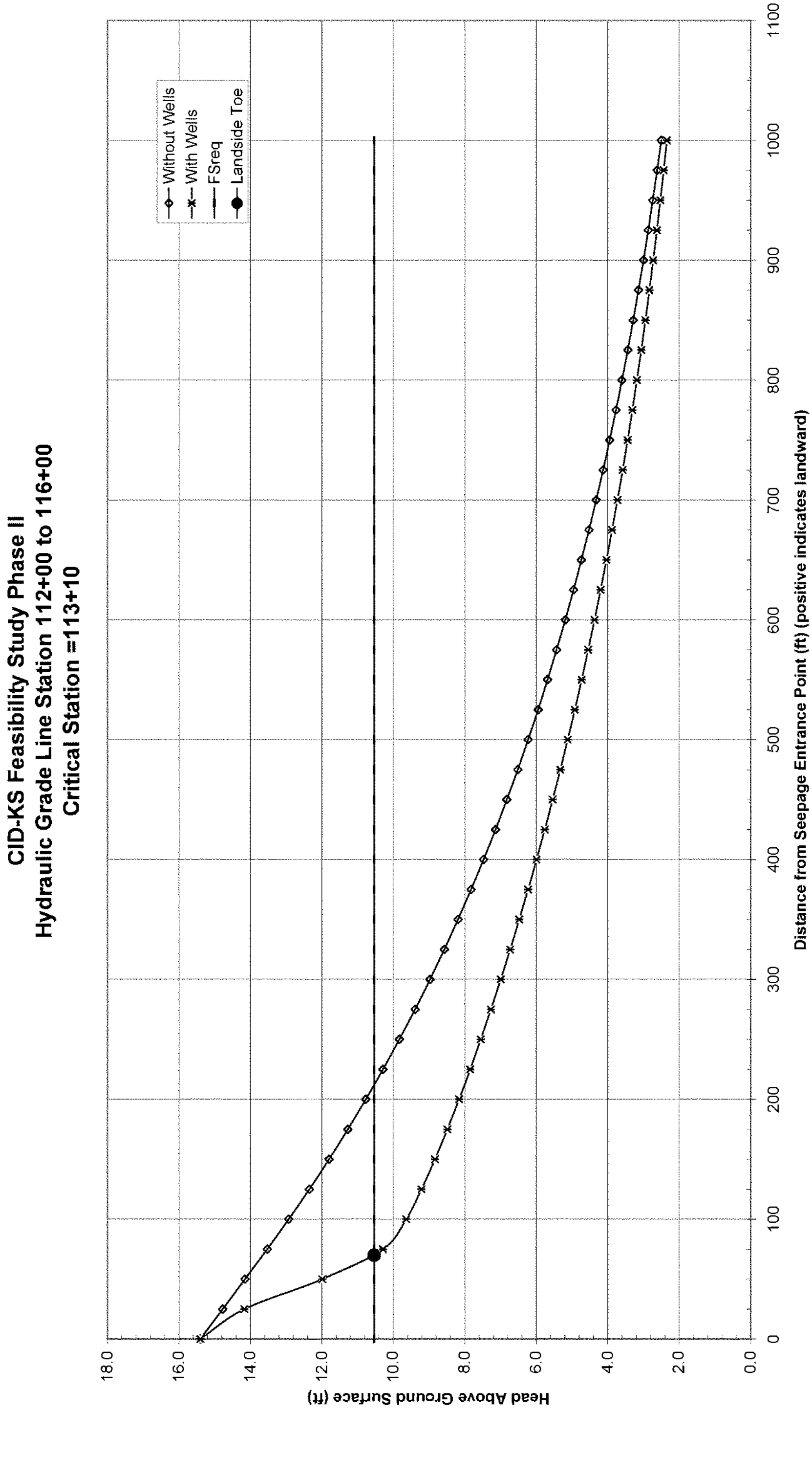
image well locations		
well	x'	y'
10	-70	11000
11	-70	11040
12	-70	11080
13	-70	11120
14	-70	11150
15	-70	11180
16	-70	11210
17	-70	11240
18	-70	11285
19	-70	11330
20	-70	11370
21	-70	11410
22	-70	11450
23	-70	11490
24	-70	11530
25	-70	11560
26	-70	11590
27	-70	11620
0	0	0
0	0	0

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	11310	15.4	0.0	15.4	10.54	1.00	0.77	1.09
2	25	11310	14.8	1.6	14.2	10.54	1.00	0.71	1.19
3	50	11310	14.2	3.7	12.0	10.54	1.00	0.60	1.41
4	75	11310	13.5	4.2	10.3	10.54	1.00	0.51	1.64
5	100	11310	12.9	4.3	9.6	10.54	1.00	0.48	1.75
6	125	11310	12.4	4.1	9.2	10.54	1.00	0.46	1.83
7	150	11310	11.8	4.0	8.8	10.54	1.00	0.44	1.91
8	175	11310	11.3	3.8	8.5	10.54	1.00	0.42	1.99
9	200	11310	10.8	3.6	8.2	10.54	1.00	0.41	2.07
10	225	11310	10.3	3.4	7.8	10.54	1.00	0.39	2.15
11	250	11310	9.8	3.3	7.6	10.54	1.00	0.38	2.23
12	275	11310	9.4	3.1	7.3	10.54	1.00	0.36	2.32
13	300	11310	9.0	3.0	7.0	10.54	1.00	0.35	2.41
14	325	11310	8.6	2.8	6.7	10.54	1.00	0.34	2.50
15	350	11310	8.2	2.7	6.5	10.54	1.00	0.32	2.60
16	375	11310	7.8	2.6	6.2	10.54	1.00	0.31	2.71
17	400	11310	7.5	2.5	6.0	10.54	1.00	0.30	2.81
18	425	11310	7.1	2.4	5.8	10.54	1.00	0.29	2.93
19	450	11310	6.8	2.3	5.5	10.54	1.00	0.28	3.04
20	475	11310	6.5	2.2	5.3	10.54	1.00	0.27	3.16
21	500	11310	6.2	2.1	5.1	10.54	1.00	0.26	3.29
22	525	11310	5.9	2.0	4.9	10.54	1.00	0.25	3.42
23	550	11310	5.7	2.0	4.7	10.54	1.00	0.24	3.56
24	575	11310	5.4	1.9	4.5	10.54	1.00	0.23	3.71
25	600	11310	5.2	1.8	4.4	10.54	1.00	0.22	3.86
26	625	11310	5.0	1.8	4.2	10.54	1.00	0.21	4.01
27	650	11310	4.7	1.7	4.0	10.54	1.00	0.20	4.18
28	675	11310	4.5	1.6	3.9	10.54	1.00	0.19	4.35
29	700	11310	4.3	1.6	3.7	10.54	1.00	0.19	4.52
30	725	11310	4.1	1.5	3.6	10.54	1.00	0.18	4.71
31	750	11310	3.9	1.5	3.4	10.54	1.00	0.17	4.90
32	775	11310	3.8	1.5	3.3	10.54	1.00	0.17	5.09
33	800	11310	3.6	1.4	3.2	10.54	1.00	0.16	5.30
34	825	11310	3.4	1.4	3.1	10.54	1.00	0.15	5.51
35	850	11310	3.3	1.3	2.9	10.54	1.00	0.15	5.72
36	875	11310	3.1	1.3	2.8	10.54	1.00	0.14	5.95
37	900	11310	3.0	1.3	2.7	10.54	1.00	0.14	6.18
38	925	11310	2.9	1.2	2.6	10.54	1.00	0.13	6.42
39	950	11310	2.7	1.2	2.5	10.54	1.00	0.13	6.67
40	975	11310	2.6	1.2	2.4	10.54	1.00	0.12	6.93
41	1000	11310	2.5	1.2	2.3	10.54	1.00	0.12	7.19

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	70	11189	13.7	4.8	9.8	10.54	1.00	0.49	1.72
2	70	11200	13.7	4.8	9.9	10.54	1.00	0.49	1.71
3	70	11211	13.7	5.7	8.9	10.54	1.00	0.45	1.88
4	70	11222	13.7	4.7	10.0	10.54	1.00	0.50	1.69
5	70	11233	13.7	4.8	9.9	10.54	1.00	0.49	1.71
6	70	11244	13.7	4.9	9.7	10.54	1.00	0.49	1.73
7	70	11255	13.7	4.3	10.3	10.54	1.00	0.52	1.63
8	70	11266	13.7	4.2	10.4	10.54	1.00	0.52	1.61
9	70	11277	13.7	4.5	10.2	10.54	1.00	0.51	1.65
10	70	11288	13.7	4.9	9.8	10.54	1.00	0.49	1.72
11	70	11299	13.7	4.2	10.4	10.54	1.00	0.52	1.61
12	70	11310	13.7	4.1	10.5	10.54	1.00	0.53	1.60
13	70	11321	13.7	4.4	10.3	10.54	1.00	0.51	1.64
14	70	11332	13.7	5.1	9.6	10.54	1.00	0.48	1.76
15	70	11343	13.7	4.3	10.4	10.54	1.00	0.52	1.63
16	70	11354	13.7	4.2	10.4	10.54	1.00	0.52	1.62
17	70	11365	13.7	4.7	10.0	10.54	1.00	0.50	1.69
18	70	11376	13.7	4.6	10.1	10.54	1.00	0.50	1.68
19	70	11387	13.7	4.2	10.4	10.54	1.00	0.52	1.62
20	70	11398	13.7	4.3	10.3	10.54	1.00	0.52	1.63
21	70	11409	13.7	5.4	9.2	10.54	1.00	0.46	1.83
22	70	11420	13.7	4.4	10.3	10.54	1.00	0.51	1.64
23	70	11431	13.7	4.2	10.4	10.54	1.00	0.52	1.62
24	70	11442	13.7	4.5	10.2	10.54	1.00	0.51	1.66
25	70	11453	13.7	4.9	9.8	10.54	1.00	0.49	1.73
26	70	11464	13.7	4.3	10.4	10.54	1.00	0.52	1.62
27	70	11475	13.7	4.3	10.4	10.54	1.00	0.52	1.62
28	70	11486	13.7	4.8	9.9	10.54	1.00	0.49	1.71
29	70	11497	13.7	4.5	10.1	10.54	1.00	0.51	1.67
30	70	11508	13.7	4.2	10.4	10.54	1.00	0.52	1.62
31	70	11519	13.7	4.4	10.3	10.54	1.00	0.51	1.64
32	70	11530	13.7	5.5	9.2	10.54	1.00	0.46	1.84
33	70	11541	13.7	4.5	10.2	10.54	1.00	0.51	1.66
34	70	11552	13.7	4.6	10.1	10.54	1.00	0.50	1.68
35	70	11563	13.7	5.0	9.7	10.54	1.00	0.48	1.74
36	70	11574	13.7	4.4	10.3	10.54	1.00	0.51	1.64
37	70	11585	13.7	4.6	10.1	10.54	1.00	0.50	1.68
38	70	11596	13.7	4.4	10.3	10.54	1.00	0.51	1.64
39	70	11607	13.7	3.9	10.7	10.54	1.00	0.54	1.57
40	70	11618	13.7	4.5	10.1	10.54	1.00	0.51	1.66
41	70	11629	13.7	3.4	11.3	10.54	1.00	0.56	1.50


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 112+00 to 116+00
Critical Station =113+10
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
0
100
200
300
400
500
600
700
800
900
1000
1100

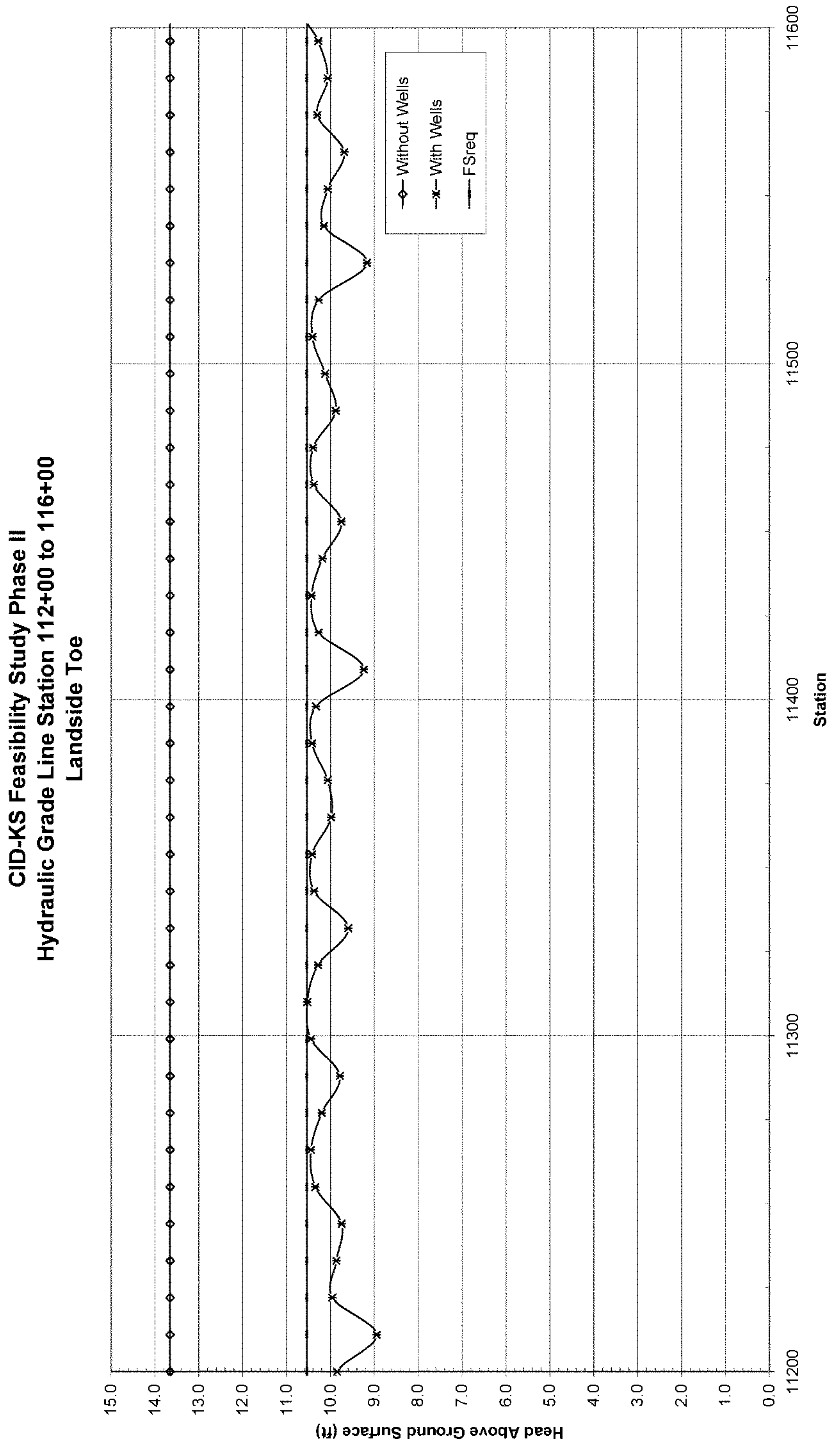

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 112+00 to 116+00
Landside Toe
Head Above Ground Surface (ft)
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
11200
11300
11400
11500
11600
Station
Without Wells
With Wells
FSreq

EXHIBIT A-4.19

Relief Well Design – Station 127+00 to 168+00

CID-KS Levee Unit 500+3 Feasibility Study

UNDERSEEPAGE ANALYSIS WITHOUT WELLS

Levee Foundation Information - 500+3

Blanket Unit Weight = 115.0 pcf (saturated)

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Blanket Elevation (msl)	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio: River Side k/k_{br}	Permeability Ratio: Land Side k/k_{bl}	Impervious Blanket Thickness (ft): River Side z_{br}	Impervious Blanket Thickness (ft): Levee z_{bc}	Impervious Blanket Thickness (ft): Land Side z_{bl}	Pervious Blanket Thickness (ft) d	Seepage Length (ft): River Side L_1	Seepage Length (ft): Levee L_2	Seepage Length (ft): Land Side L_3	Factor c_r	Factor c_l	Effective Seepage Length (ft): River Side x_1	Effective Seepage Length (ft): Land Side x_3	Head at Toe (ft) h_0	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Factor of Safety for Piping (i_c/i_o)	Remarks
127+00 to 132+00	769.6	740	750	728	728	750	670	13.5	400	400	12	17	22	58	0	70	1000	0.001895	0.001400	0	714	12.30	0.56	0.84	1.51	fill effects
132+00 to 134+00	769.8	740	755	730	730	756	670	14.8	400	400	10	17.5	25	60	0	70	1000	0.002041	0.001291	0	775	13.57	0.54	0.84	1.55	
134+00 to 138+00	769.9	740	755	730	730	755	670	14.9	600	600	10	17.5	25	60	0	70	800	0.001667	0.001054	0	1380	14.13	0.57	0.84	1.49	Bluff blocks seepage distance
138+00 to 143+00	770.1	740	753	727	727	753	670	17.1	400	400	13	19.5	26	57	0	70	700	0.001837	0.001299	0	1068	16.05	0.62	0.84	1.37	Bluff blocks seepage distance
143+00 to 150+00	770.5	740	752	725	725	748	670	18.5	300	300	15	19	23	55	0	70	520	0.002010	0.001623	0	895	17.16	0.75	0.84	1.13	Bluff blocks seepage distance
150+00 to 153+00	770.9	740	752	725	725	748	670	18.9	700	700	15	19	23	55	0	70	465	0.001319	0.001063	0	1951	18.25	0.79	0.84	1.06	Bluff blocks seepage distance
153+00 to 159+00	771.2	740	753	725	725	748	670	18.2	600	600	15	19	23	55	0	70	415	0.001421	0.001148	0	1965	17.57	0.78	0.84	1.10	Bluff blocks seepage distance
159+00 to 163+00	771.5	740	754	725	725	746	670	17.5	300	800	15	18	21	55	0	70	325	0.001231	0.001040	0	2951	17.09	0.81	0.84	1.04	Bluff blocks seepage distance
163+00 to 167+00	771.7	740	755	725	725	750	670	16.7	800	800	15	20	25	55	0	70	245	0.001231	0.000953	0	4571	16.45	0.66	0.84	1.28	Bluff blocks seepage distance
167+00 to 168+00	771.7	741	755	740	740	750	740	16.7	300	800	1	5.5	10	0	0	30	245	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.84	#DIV/0!	Bluff blocks seepage distance

N500+3 Relief Well System Summary Station 127+00 to 168+00

Well	Distance From Seepage Entrance (ft)	Station	Discharge Elevation (ft)	0.8*Qw (cfs)	Qw (cfs)
1	70	127+00	759	0.90	1.13
2	70	128+00	759	0.86	1.08
3	70	129+00	759	0.85	1.06
4	70	130+00	759	0.85	1.06
5	70	131+00	759	0.97	1.21
6	70	132+00	759	0.93	1.16
7	70	133+00	759	1.03	1.29
8	70	134+00	759	0.99	1.23
9	70	134+95	759	0.98	1.23
10	70	135+90	759	1.03	1.28
11	70	136+45	759	0.77	0.97
12	70	137+05	759	0.77	0.96
13	70	137+70	759	0.76	0.95
14	70	138+30	759	0.75	0.94
15	70	138+90	759	0.75	0.94
16	70	139+50	759	0.75	0.93
17	70	140+10	759	0.75	0.93
18	70	140+70	759	0.75	0.93
19	70	141+30	759	0.75	0.93
20	70	141+90	759	0.76	0.95
21	70	142+50	759	0.70	0.87
22	70	142+75	759	0.69	0.86
23	70	143+00	759	0.74	0.92
24	70	143+25	752	0.97	1.21
25	70	143+50	752	1.01	1.26
26	70	144+00	752	1.08	1.35
27	70	144+50	752	1.10	1.38
28	70	145+00	752	1.12	1.39
29	70	145+50	752	1.12	1.40
30	70	146+00	752	1.13	1.41
31	70	146+50	752	1.13	1.41
32	70	147+00	752	1.13	1.41
33	70	147+50	752	1.13	1.42
34	70	148+00	752	1.14	1.42
35	70	148+50	752	1.14	1.43
36	70	149+00	752	1.29	1.61
37	70	149+50	752	1.11	1.39
38	70	149+75	752	1.10	1.38
39	70	150+00	752	1.19	1.48
48	450	150+00	752	0.91	1.14
40	70	150+50	752	1.06	1.32
41	70	150+75	752	1.06	1.33
42	70	151+25	752	1.09	1.36
49	450	151+50	752	0.88	1.10
43	70	151+65	752	1.09	1.36
44	70	152+05	752	1.10	1.38
45	70	152+45	752	1.33	1.66
46	70	152+85	752	1.20	1.50
50	450	153+00	752	0.99	1.24
47	70	153+10	752	1.22	1.52
51	70	153+70	753	1.15	1.44
52	70	154+20	753	1.14	1.42
53	70	154+70	753	1.13	1.41
54	70	155+20	753	1.12	1.40
55	70	155+70	753	1.12	1.40
63	400	156+00	753	0.89	1.11
56	70	156+20	753	1.12	1.40
57	70	156+70	753	1.13	1.41
58	70	157+20	753	1.13	1.42
59	70	157+70	753	1.34	1.67
60	70	158+20	753	1.21	1.52
61	70	158+70	753	1.18	1.47
62	70	159+00	753	1.24	1.55
64	400	159+00	753	0.96	1.20
65	70	159+50	754	1.03	1.29
66	70	159+90	754	1.01	1.26
67	70	160+30	754	1.00	1.25
68	70	160+70	754	0.99	1.24
69	70	161+10	754	0.99	1.24
70	70	161+50	754	1.00	1.25
71	70	161+90	754	1.01	1.27
72	70	162+30	754	1.28	1.60
73	70	162+70	754	1.18	1.48
74	70	163+00	754	1.20	1.49
75	300	161+00	754	1.09	1.36
76	300	163+00	754	1.01	1.27
77	70	163+75	755	1.20	1.50
78	70	164+50	755	1.23	1.53
79	70	165+25	755	1.24	1.55
80	70	166+00	755	1.25	1.56
81	70	166+75	755	1.24	1.55
82	70	167+25	755	1.24	1.55
83	70	167+90	755	1.24	1.55
				86.16	**107.70**

some wells discharge at elevation just landside of wall (759)
some wells discharge at elevation landward of wall and a discharge details is ne
needed some wells along bluff

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	58	ft
h_o =	12.30	ft
TOL =	769.5	ft elevation
Landside =	756.0	ft elevation
Bottom Blanket =	728	ft elevation
blanket =	22.0	ft
z, Landside Toe =	70	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	6.54	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
1	70	12700	759.0	0.90	0.29	1.00
2	70	12800	759.0	0.86	0.27	1.00
3	70	12900	759.0	0.85	0.27	1.00
4	70	13000	759.0	0.85	0.27	1.00
5	70	13100	759.0	0.86	0.27	1.00
6	70	13200	759.0	0.90	0.29	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

1.00 <---Input H_w AVG after any changes are made to well parameters

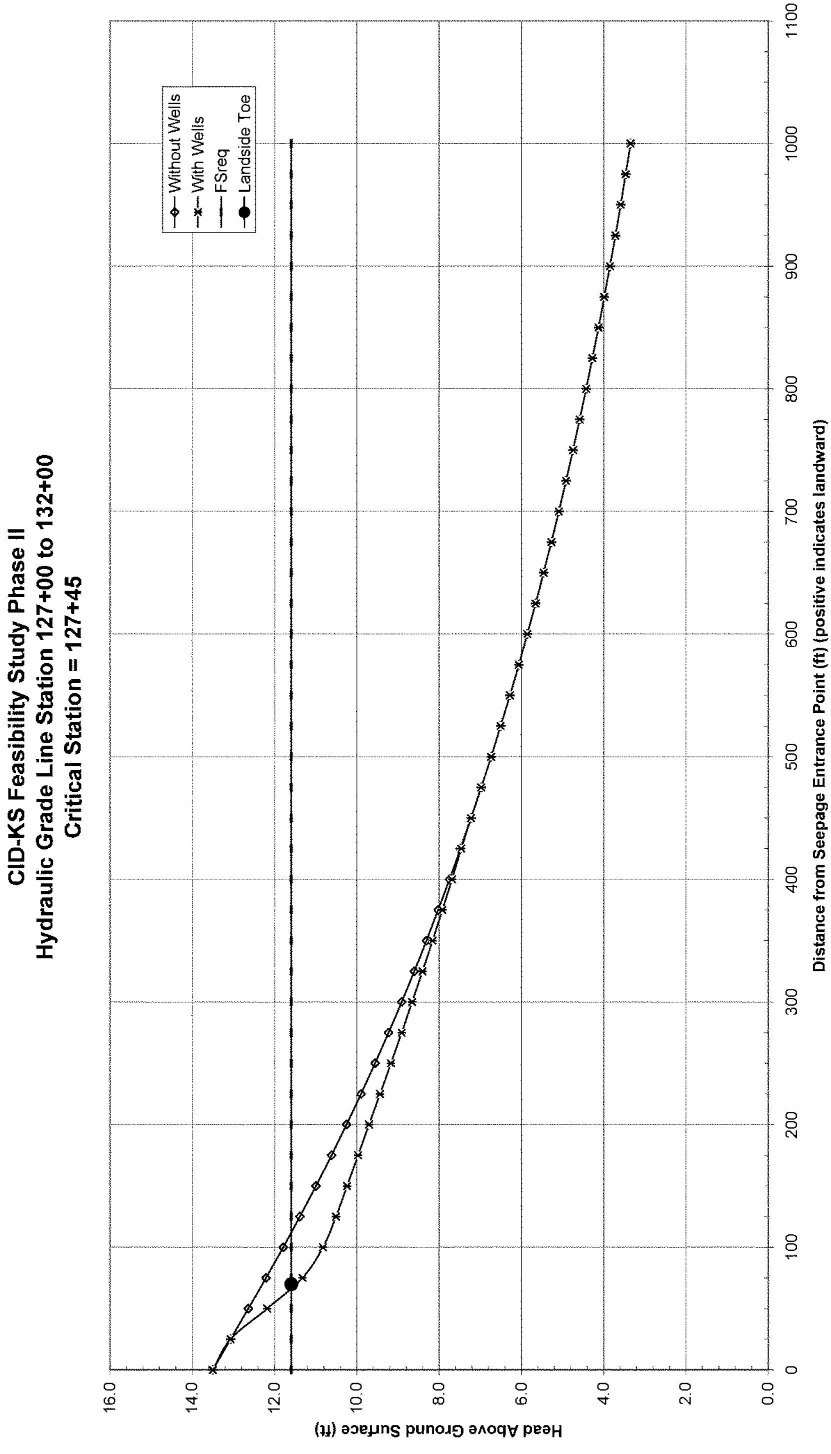
image well locations		
well	x'	y'
1	-70	12700
2	-70	12800
3	-70	12900
4	-70	13000
5	-70	13100
6	-70	13200
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

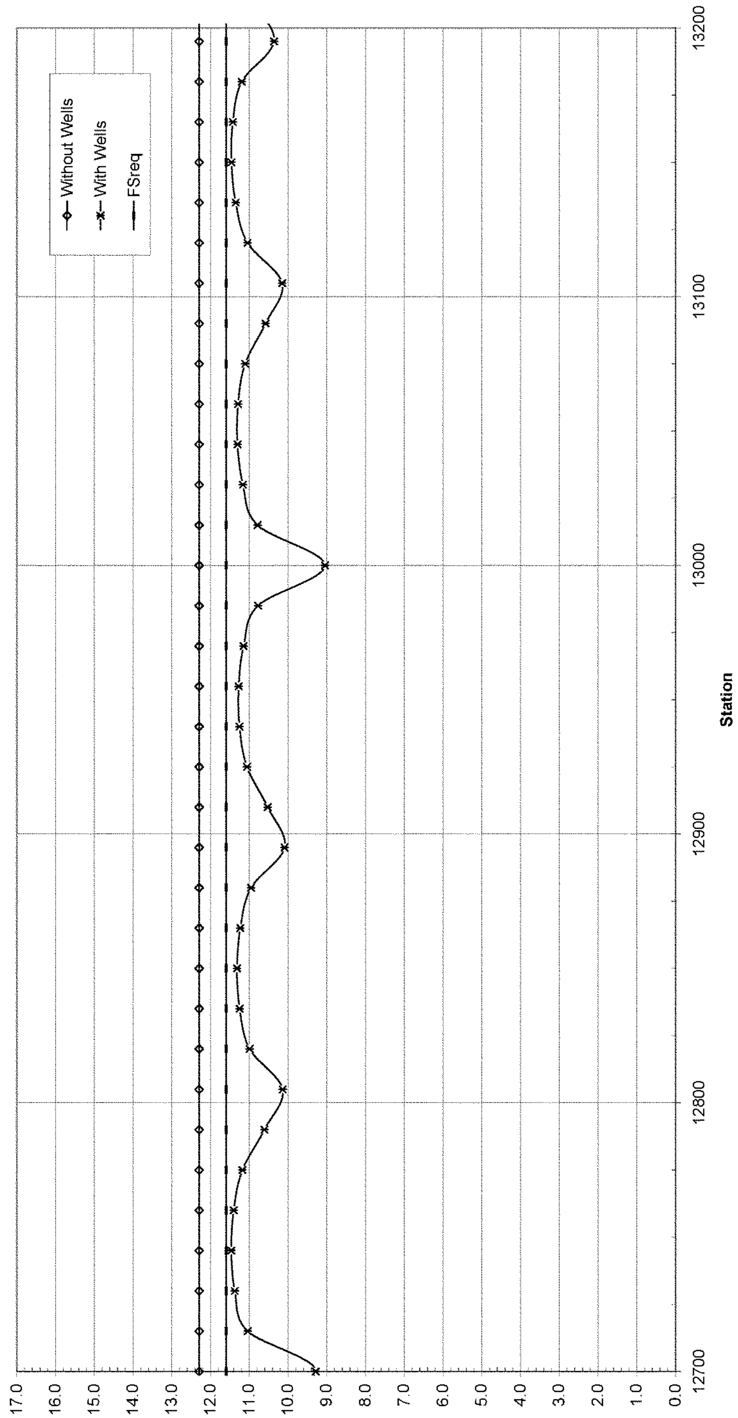
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_e (ft)	H_w (ft)	i	FS_i
1	0	12745	13.5	0.0	13.5	11.59	1.00	0.61	1.37
2	25	12745	13.1	0.8	13.1	11.59	1.00	0.59	1.42
3	50	12745	12.6	1.5	12.2	11.59	1.00	0.55	1.52
4	75	12745	12.2	1.9	11.3	11.59	1.00	0.51	1.64
5	100	12745	11.8	2.0	10.8	11.59	1.00	0.49	1.71
6	125	12745	11.4	1.9	10.5	11.59	1.00	0.48	1.76
7	150	12745	11.0	1.8	10.2	11.59	1.00	0.47	1.81
8	175	12745	10.6	1.6	10.0	11.59	1.00	0.45	1.86
9	200	12745	10.2	1.5	9.7	11.59	1.00	0.44	1.91
10	225	12745	9.9	1.5	9.4	11.59	1.00	0.43	1.97
11	250	12745	9.6	1.4	9.2	11.59	1.00	0.42	2.02
12	275	12745	9.2	1.3	8.9	11.59	1.00	0.41	2.08
13	300	12745	8.9	1.3	8.7	11.59	1.00	0.39	2.14
14	325	12745	8.6	1.2	8.4	11.59	1.00	0.38	2.21
15	350	12745	8.3	1.1	8.2	11.59	1.00	0.37	2.27
16	375	12745	8.0	1.1	7.9	11.59	1.00	0.36	2.34
17	400	12745	7.7	1.1	7.7	11.59	1.00	0.35	2.41
18	425	12745	7.5	1.0	7.5	11.59	1.00	0.34	2.49
19	450	12745	7.2	1.0	7.2	11.59	1.00	0.33	2.57
20	475	12745	7.0	0.9	7.0	11.59	1.00	0.32	2.66
21	500	12745	6.7	0.9	6.7	11.59	1.00	0.31	2.75
22	525	12745	6.5	0.9	6.5	11.59	1.00	0.30	2.85
23	550	12745	6.3	0.9	6.3	11.59	1.00	0.29	2.96
24	575	12745	6.1	0.8	6.1	11.59	1.00	0.28	3.06
25	600	12745	5.9	0.8	5.9	11.59	1.00	0.27	3.17
26	625	12745	5.7	0.8	5.7	11.59	1.00	0.26	3.28
27	650	12745	5.5	0.8	5.5	11.59	1.00	0.25	3.40
28	675	12745	5.3	0.7	5.3	11.59	1.00	0.24	3.52
29	700	12745	5.1	0.7	5.1	11.59	1.00	0.23	3.64
30	725	12745	4.9	0.7	4.9	11.59	1.00	0.22	3.77
31	750	12745	4.7	0.7	4.7	11.59	1.00	0.22	3.91
32	775	12745	4.6	0.7	4.6	11.59	1.00	0.21	4.05
33	800	12745	4.4	0.6	4.4	11.59	1.00	0.20	4.19
34	825	12745	4.3	0.6	4.3	11.59	1.00	0.19	4.34
35	850	12745	4.1	0.6	4.1	11.59	1.00	0.19	4.49
36	875	12745	4.0	0.6	4.0	11.59	1.00	0.18	4.65
37	900	12745	3.8	0.6	3.8	11.59	1.00	0.17	4.82
38	925	12745	3.7	0.6	3.7	11.59	1.00	0.17	4.99
39	950	12745	3.6	0.5	3.6	11.59	1.00	0.16	5.17
40	975	12745	3.5	0.5	3.5	11.59	1.00	0.16	5.35
41	1000	12745	3.3	0.5	3.3	11.59	1.00	0.15	5.54

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_e (ft)	H_w (ft)	i	FS_i
1	70	12685	12.3	2.1	11.2	11.59	1.00	0.51	1.65
2	70	12700	12.3	4.0	9.3	11.59	1.00	0.42	2.00
3	70	12715	12.3	2.3	11.0	11.59	1.00	0.50	1.68
4	70	12730	12.3	1.9	11.4	11.59	1.00	0.52	1.63
5	70	12745	12.3	1.8	11.5	11.59	1.00	0.52	1.62
6	70	12760	12.3	1.9	11.4	11.59	1.00	0.52	1.63
7	70	12775	12.3	2.1	11.2	11.59	1.00	0.51	1.66
8	70	12790	12.3	2.7	10.6	11.59	1.00	0.48	1.75
9	70	12805	12.3	3.2	10.1	11.59	1.00	0.46	1.83
10	70	12820	12.3	2.3	11.0	11.59	1.00	0.50	1.69
11	70	12835	12.3	2.0	11.2	11.59	1.00	0.51	1.65
12	70	12850	12.3	2.0	11.3	11.59	1.00	0.51	1.64
13	70	12865	12.3	2.1	11.2	11.59	1.00	0.51	1.65
14	70	12880	12.3	2.3	11.0	11.59	1.00	0.50	1.69
15	70	12895	12.3	3.2	10.1	11.59	1.00	0.46	1.84
16	70	12910	12.3	2.8	10.5	11.59	1.00	0.48	1.76
17	70	12925	12.3	2.2	11.1	11.59	1.00	0.50	1.68
18	70	12940	12.3	2.0	11.3	11.59	1.00	0.51	1.65
19	70	12955	12.3	2.0	11.3	11.59	1.00	0.51	1.64
20	70	12970	12.3	2.1	11.1	11.59	1.00	0.51	1.66
21	70	12985	12.3	2.5	10.8	11.59	1.00	0.49	1.72
22	70	13000	12.3	4.2	9.0	11.59	1.00	0.41	2.05
23	70	13015	12.3	2.5	10.8	11.59	1.00	0.49	1.72
24	70	13030	12.3	2.1	11.2	11.59	1.00	0.51	1.66
25	70	13045	12.3	2.0	11.3	11.59	1.00	0.51	1.64
26	70	13060	12.3	2.0	11.3	11.59	1.00	0.51	1.64
27	70	13075	12.3	2.2	11.1	11.59	1.00	0.50	1.67
28	70	13090	12.3	2.7	10.6	11.59	1.00	0.48	1.75
29	70	13105	12.3	3.1	10.2	11.59	1.00	0.46	1.83
30	70	13120	12.3	2.2	11.0	11.59	1.00	0.50	1.68
31	70	13135	12.3	1.9	11.3	11.59	1.00	0.52	1.63
32	70	13150	12.3	1.8	11.5	11.59	1.00	0.52	1.62
33	70	13165	12.3	1.9	11.4	11.59	1.00	0.52	1.62
34	70	13180	12.3	2.1	11.2	11.59	1.00	0.51	1.66
35	70	13195	12.3	2.9	10.4	11.59	1.00	0.47	1.79
36	70	13210	12.3	2.4	10.9	11.59	1.00	0.50	1.70
37	70	13225	12.3	1.7	11.6	11.59	1.00	0.53	1.60
38	70	13240	12.3	1.3	12.0	11.59	1.00	0.54	1.55
39	70	13255	12.3	1.1	12.2	11.59	1.00	0.56	1.52
40	70	13270	12.3	0.9	12.3	11.59	1.00	0.56	1.51
41	70	13285	12.3	0.8	12.3	11.59	1.00	0.56	1.51


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 127+00 to 132+00
Critical Station = 127+45
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 127+00 to 132+00
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	60	ft
h_0 =	13.57	ft
TOL =	769.8	ft elevation
Landside =	755.0	ft elevation
Bottom Blanket =	730	ft elevation
blanket =	25.0	ft
z, Landside Toe =	70	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	4.75	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
5	70	13100	759.0	0.97	0.31	1.00
6	70	13200	759.0	0.93	0.30	1.00
7	70	13300	759.0	0.93	0.30	1.00
8	70	13400	759.0	0.97	0.31	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

<--Input H_w AVG after any changes are made to well parameters

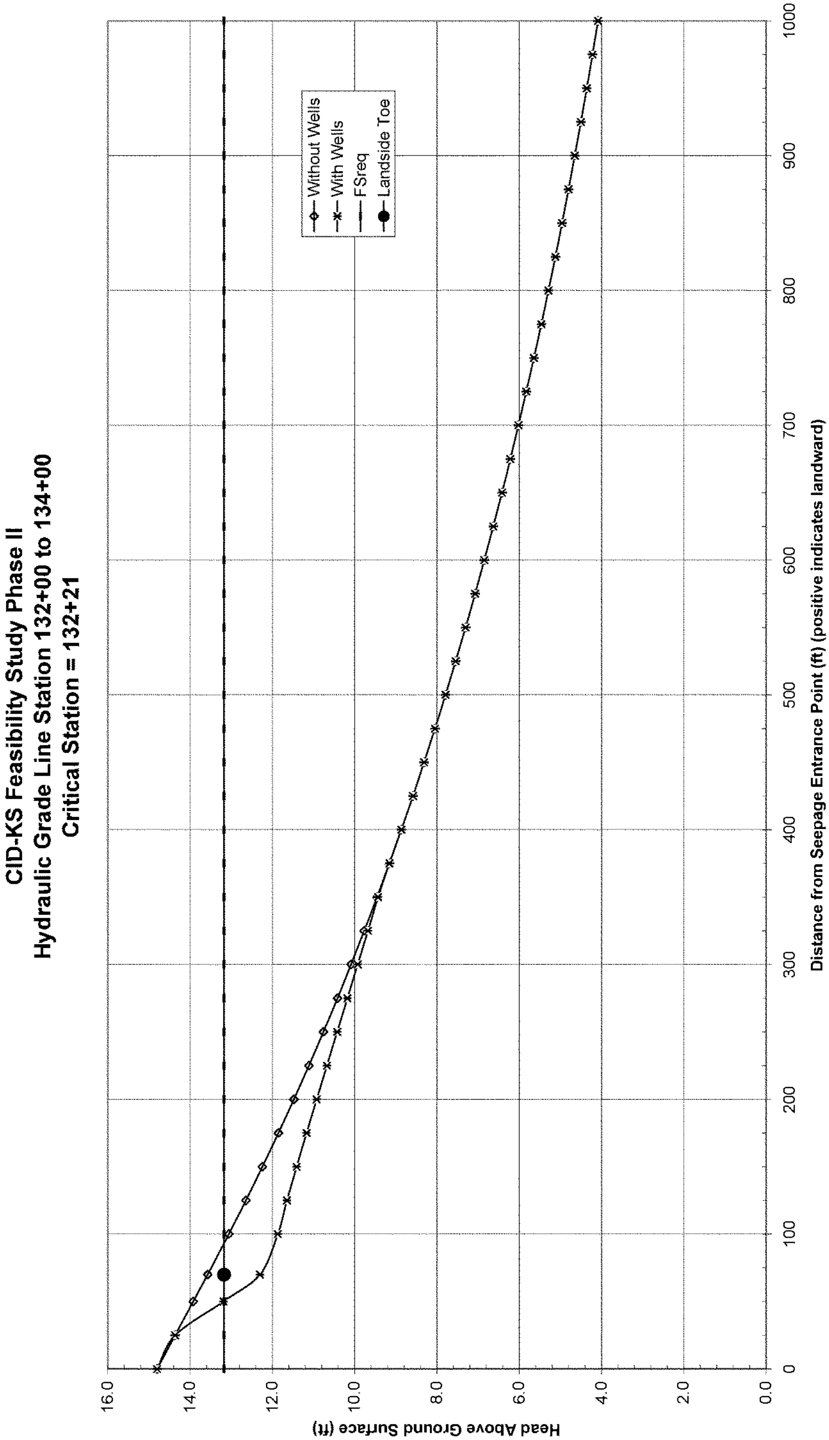
image well locations		
well	x'	y'
5	-70	13100
6	-70	13200
7	-70	13300
8	-70	13400
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

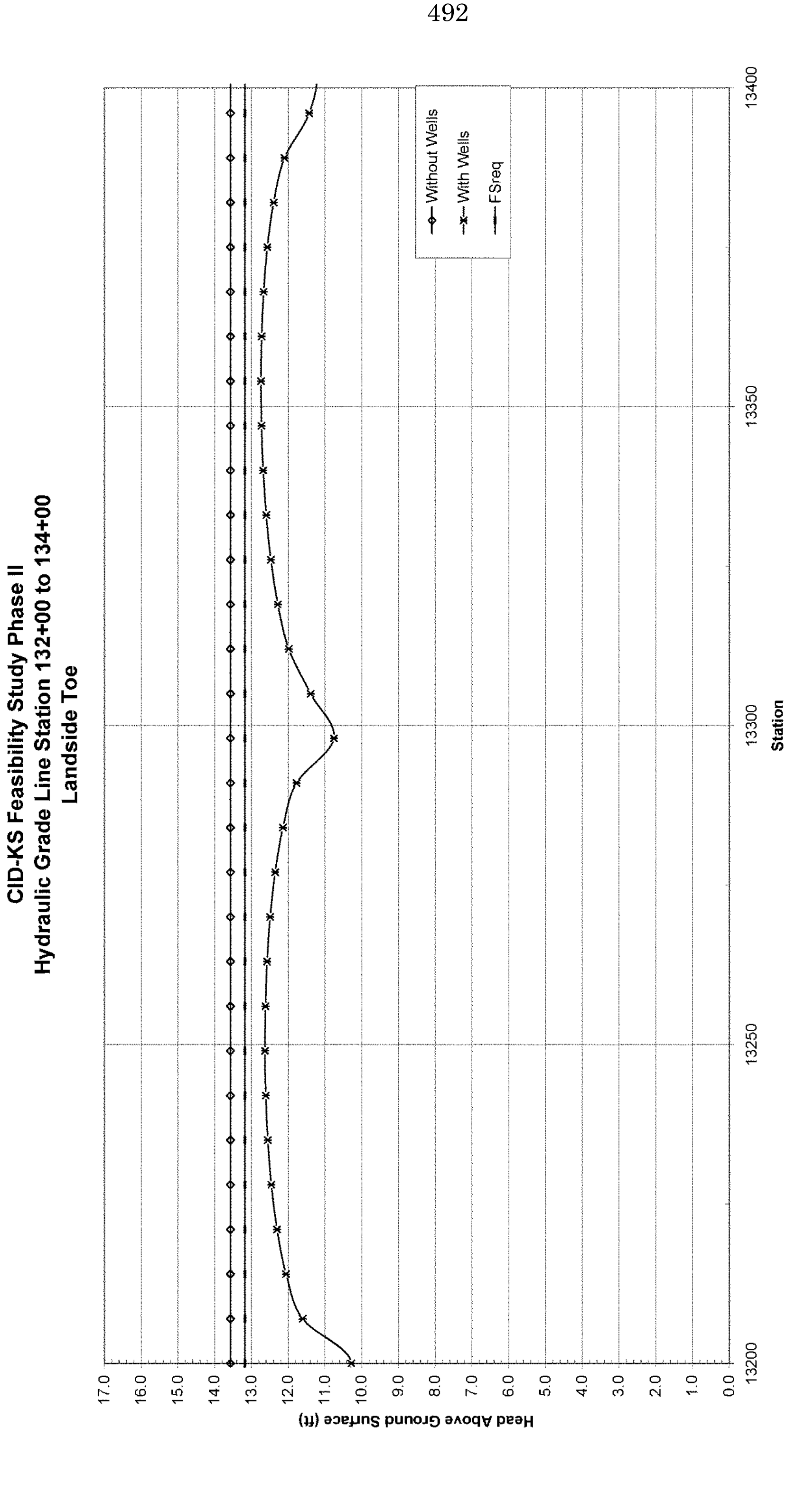
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_b (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	13221	14.8	0.0	14.8	13.17	1.00	0.59	1.42
2	25	13221	14.4	0.9	14.4	13.17	1.00	0.57	1.47
3	50	13221	13.9	1.7	13.2	13.17	1.00	0.53	1.60
4	70	13221	13.6	2.3	12.3	13.17	1.00	0.49	1.71
5	100	13221	13.1	2.2	11.9	13.17	1.00	0.47	1.78
6	125	13221	12.6	2.0	11.6	13.17	1.00	0.47	1.81
7	150	13221	12.2	1.8	11.4	13.17	1.00	0.46	1.85
8	175	13221	11.9	1.7	11.2	13.17	1.00	0.45	1.89
9	200	13221	11.5	1.5	10.9	13.17	1.00	0.44	1.93
10	225	13221	11.1	1.4	10.7	13.17	1.00	0.43	1.97
11	250	13221	10.8	1.3	10.4	13.17	1.00	0.42	2.02
12	275	13221	10.4	1.2	10.2	13.17	1.00	0.41	2.07
13	300	13221	10.1	1.2	9.9	13.17	1.00	0.40	2.12
14	325	13221	9.8	1.1	9.7	13.17	1.00	0.39	2.18
15	350	13221	9.5	1.0	9.4	13.17	1.00	0.38	2.23
16	375	13221	9.2	1.0	9.2	13.17	1.00	0.37	2.30
17	400	13221	8.9	0.9	8.9	13.17	1.00	0.35	2.38
18	425	13221	8.6	0.9	8.6	13.17	1.00	0.34	2.46
19	450	13221	8.3	0.8	8.3	13.17	1.00	0.33	2.54
20	475	13221	8.0	0.8	8.0	13.17	1.00	0.32	2.62
21	500	13221	7.8	0.7	7.8	13.17	1.00	0.31	2.70
22	525	13221	7.5	0.7	7.5	13.17	1.00	0.30	2.79
23	550	13221	7.3	0.7	7.3	13.17	1.00	0.29	2.89
24	575	13221	7.1	0.7	7.1	13.17	1.00	0.28	2.98
25	600	13221	6.8	0.6	6.8	13.17	1.00	0.27	3.08
26	625	13221	6.6	0.6	6.6	13.17	1.00	0.27	3.18
27	650	13221	6.4	0.6	6.4	13.17	1.00	0.26	3.28
28	675	13221	6.2	0.6	6.2	13.17	1.00	0.25	3.39
29	700	13221	6.0	0.5	6.0	13.17	1.00	0.24	3.50
30	725	13221	5.8	0.5	5.8	13.17	1.00	0.23	3.62
31	750	13221	5.6	0.5	5.6	13.17	1.00	0.23	3.74
32	775	13221	5.5	0.5	5.5	13.17	1.00	0.22	3.86
33	800	13221	5.3	0.5	5.3	13.17	1.00	0.21	3.98
34	825	13221	5.1	0.5	5.1	13.17	1.00	0.20	4.11
35	850	13221	5.0	0.5	5.0	13.17	1.00	0.20	4.25
36	875	13221	4.8	0.4	4.8	13.17	1.00	0.19	4.39
37	900	13221	4.6	0.4	4.6	13.17	1.00	0.19	4.53
38	925	13221	4.5	0.4	4.5	13.17	1.00	0.18	4.68
39	950	13221	4.4	0.4	4.4	13.17	1.00	0.17	4.84
40	975	13221	4.2	0.4	4.2	13.17	1.00	0.17	4.99
41	1000	13221	4.1	0.4	4.1	13.17	1.00	0.16	5.16

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_b (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	70	13193	13.6	3.0	11.6	13.17	1.00	0.46	1.81
2	70	13200	13.6	4.3	10.3	13.17	1.00	0.41	2.05
3	70	13207	13.6	3.0	11.6	13.17	1.00	0.46	1.82
4	70	13214	13.6	2.5	12.1	13.17	1.00	0.48	1.75
5	70	13221	13.6	2.3	12.3	13.17	1.00	0.49	1.71
6	70	13228	13.6	2.1	12.5	13.17	1.00	0.50	1.69
7	70	13235	13.6	2.0	12.6	13.17	1.00	0.50	1.68
8	70	13242	13.6	2.0	12.6	13.17	1.00	0.50	1.67
9	70	13249	13.6	1.9	12.6	13.17	1.00	0.51	1.67
10	70	13256	13.6	2.0	12.6	13.17	1.00	0.50	1.67
11	70	13263	13.6	2.0	12.6	13.17	1.00	0.50	1.68
12	70	13270	13.6	2.1	12.5	13.17	1.00	0.50	1.69
13	70	13277	13.6	2.2	12.4	13.17	1.00	0.49	1.71
14	70	13284	13.6	2.4	12.1	13.17	1.00	0.48	1.74
15	70	13291	13.6	2.8	11.8	13.17	1.00	0.47	1.79
16	70	13298	13.6	3.8	10.8	13.17	1.00	0.43	1.96
17	70	13305	13.6	3.2	11.4	13.17	1.00	0.46	1.85
18	70	13312	13.6	2.6	12.0	13.17	1.00	0.48	1.76
19	70	13319	13.6	2.3	12.3	13.17	1.00	0.49	1.72
20	70	13326	13.6	2.1	12.5	13.17	1.00	0.50	1.69
21	70	13333	13.6	2.0	12.6	13.17	1.00	0.50	1.67
22	70	13340	13.6	1.9	12.7	13.17	1.00	0.51	1.66
23	70	13347	13.6	1.8	12.7	13.17	1.00	0.51	1.66
24	70	13354	13.6	1.8	12.7	13.17	1.00	0.51	1.65
25	70	13361	13.6	1.9	12.7	13.17	1.00	0.51	1.66
26	70	13368	13.6	1.9	12.7	13.17	1.00	0.51	1.66
27	70	13375	13.6	2.0	12.6	13.17	1.00	0.50	1.68
28	70	13382	13.6	2.2	12.4	13.17	1.00	0.50	1.70
29	70	13389	13.6	2.5	12.1	13.17	1.00	0.48	1.74
30	70	13396	13.6	3.1	11.4	13.17	1.00	0.46	1.84
31	70	13403	13.6	3.3	11.3	13.17	1.00	0.45	1.87
32	70	13410	13.6	2.4	12.2	13.17	1.00	0.49	1.73
33	70	13417	13.6	2.0	12.6	13.17	1.00	0.50	1.68
34	70	13424	13.6	1.7	12.8	13.17	1.00	0.51	1.64
35	70	13431	13.6	1.5	13.1	13.17	1.00	0.52	1.61
36	70	13438	13.6	1.4	13.2	13.17	1.00	0.53	1.59
37	70	13445	13.6	1.2	13.4	13.17	1.00	0.53	1.58
38	70	13452	13.6	1.1	13.5	13.17	1.00	0.54	1.57
39	70	13459	13.6	1.0	13.6	13.17	1.00	0.54	1.55
40	70	13466	13.6	0.9	13.6	13.17	1.00	0.54	1.55
41	70	13473	13.6	0.9	13.6	13.17	1.00	0.54	1.55


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 132+00 to 134+00
Critical Station = 132+21
Without Wells
With Wells
FSreq
Landside Toe
Head Above Ground Surface (ft)
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
Distance from Seepage Entrance Point (ft) (positive indicates landward)



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 132+00 to 134+00
Landside Toe
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
13200
13250
13300
13350
13400
Station
Without Wells
With Wells
FSreq

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	60	ft
h_p =	14.18	ft
TOL =	769.9	ft elevation
Landside =	755.0	ft elevation
Bottom Blanket =	730	ft elevation
blanket =	25.0	ft
z, Landside Toe =	70	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	5.03	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
7	70	13300	759.0	1.03	0.33	1.00
8	70	13400	759.0	0.99	0.31	1.00
9	70	13495	759.0	0.98	0.31	1.00
10	70	13590	759.0	1.03	0.33	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00

image well locations		
well	x'	y'
7	-70	13300
8	-70	13400
9	-70	13495
10	-70	13590
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

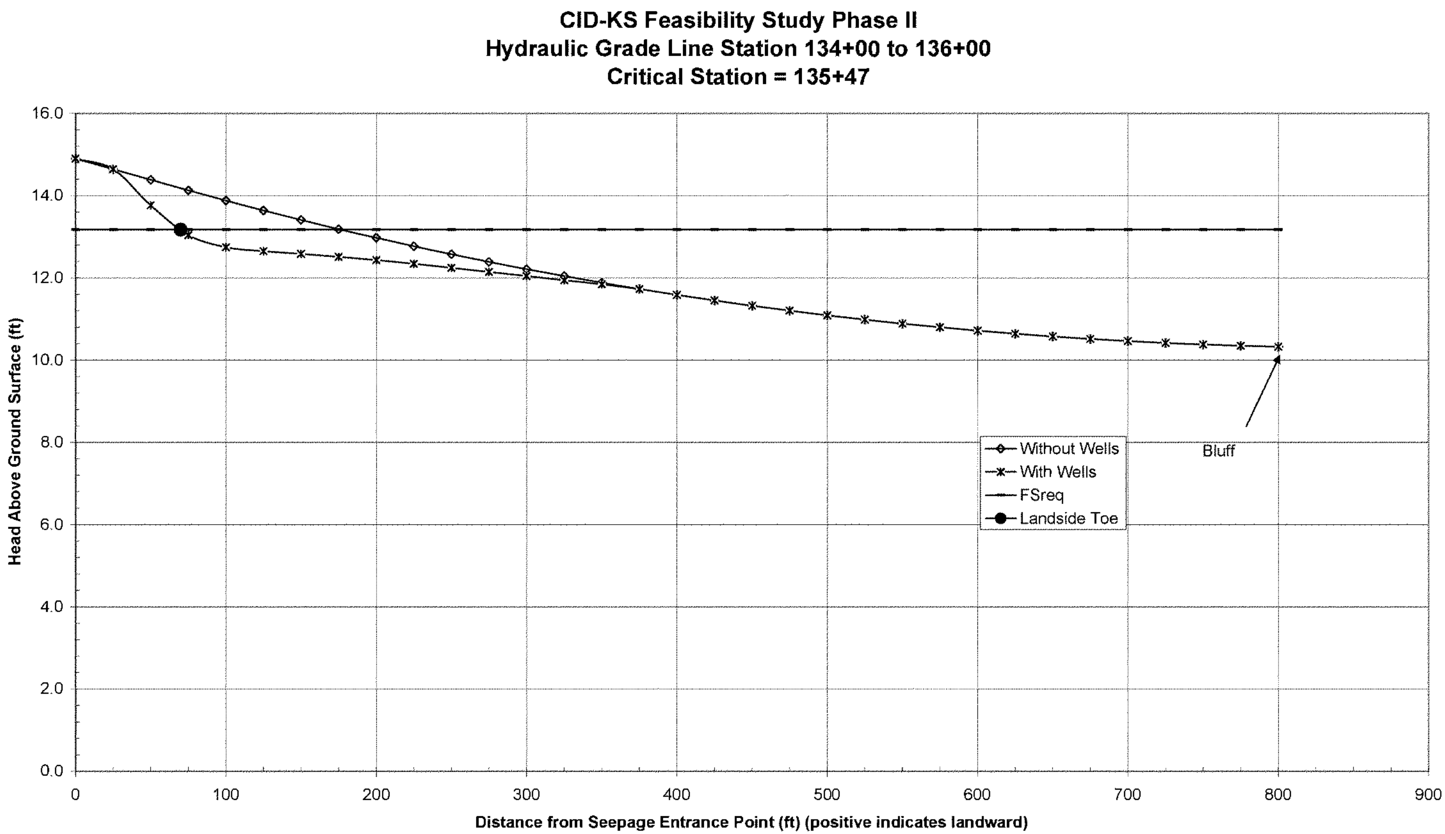
1.00 <---**Input H_w AVG after any changes are made to well parameters**

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

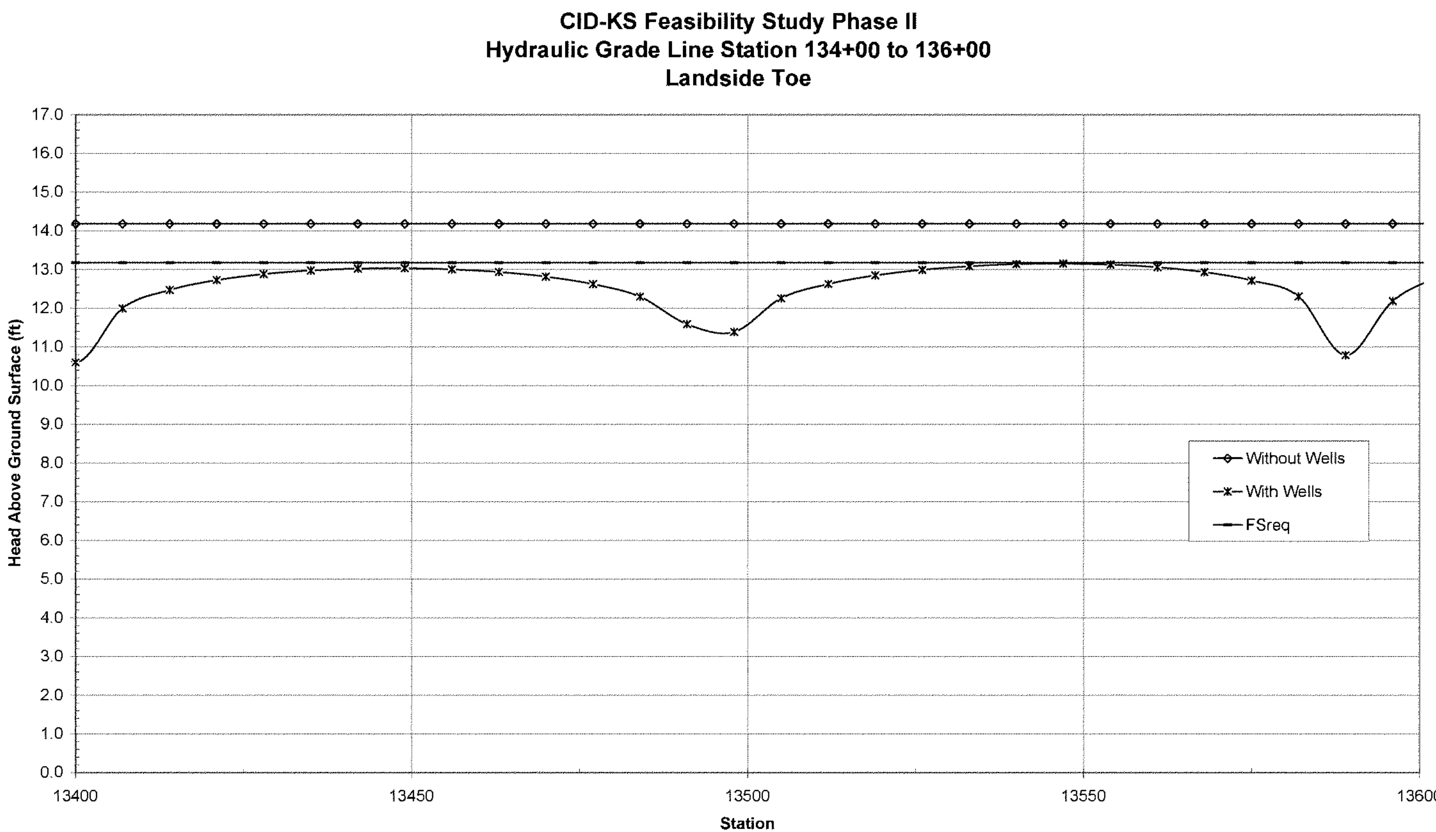
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	13547	14.9	0.0	14.9	13.17	1.00	0.60	1.41
2	25	13547	14.6	0.9	14.6	13.17	1.00	0.59	1.44
3	50	13547	14.4	1.6	13.8	13.17	1.00	0.55	1.53
4	75	13547	14.1	2.1	13.0	13.17	1.00	0.52	1.62
5	100	13547	13.9	2.1	12.7	13.17	1.00	0.51	1.65
6	125	13547	13.6	2.0	12.6	13.17	1.00	0.51	1.67
7	150	13547	13.4	1.8	12.6	13.17	1.00	0.50	1.67
8	175	13547	13.2	1.7	12.5	13.17	1.00	0.50	1.68
9	200	13547	13.0	1.5	12.4	13.17	1.00	0.50	1.70
10	225	13547	12.8	1.4	12.3	13.17	1.00	0.49	1.71
11	250	13547	12.6	1.3	12.2	13.17	1.00	0.49	1.72
12	275	13547	12.4	1.2	12.1	13.17	1.00	0.49	1.74
13	300	13547	12.2	1.2	12.0	13.17	1.00	0.48	1.75
14	325	13547	12.0	1.1	11.9	13.17	1.00	0.48	1.76
15	350	13547	11.9	1.0	11.8	13.17	1.00	0.47	1.78
16	375	13547	11.7	1.0	11.7	13.17	1.00	0.47	1.80
17	400	13547	11.6	0.9	11.6	13.17	1.00	0.46	1.82
18	425	13547	11.5	0.9	11.5	13.17	1.00	0.46	1.84
19	450	13547	11.3	0.8	11.3	13.17	1.00	0.45	1.86
20	475	13547	11.2	0.8	11.2	13.17	1.00	0.45	1.88
21	500	13547	11.1	0.8	11.1	13.17	1.00	0.44	1.90
22	525	13547	11.0	0.7	11.0	13.17	1.00	0.44	1.92
23	550	13547	10.9	0.7	10.9	13.17	1.00	0.44	1.94
24	575	13547	10.8	0.7	10.8	13.17	1.00	0.43	1.95
25	600	13547	10.7	0.7	10.7	13.17	1.00	0.43	1.97
26	625	13547	10.6	0.6	10.6	13.17	1.00	0.43	1.98
27	650	13547	10.6	0.6	10.6	13.17	1.00	0.42	1.99
28	675	13547	10.5	0.6	10.5	13.17	1.00	0.42	2.00
29	700	13547	10.5	0.6	10.5	13.17	1.00	0.42	2.01
30	725	13547	10.4	0.6	10.4	13.17	1.00	0.42	2.02
31	750	13547	10.4	0.5	10.4	13.17	1.00	0.42	2.03
32	775	13547	10.3	0.5	10.3	13.17	1.00	0.41	2.04
33	800	13547	10.3	0.5	10.3	13.17	1.00	0.41	2.04

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	70	13393	14.2	3.2	12.0	13.17	1.00	0.48	1.75
2	70	13400	14.2	4.6	10.6	13.17	1.00	0.42	1.99
3	70	13407	14.2	3.2	12.0	13.17	1.00	0.48	1.76
4	70	13414	14.2	2.7	12.5	13.17	1.00	0.50	1.69
5	70	13421	14.2	2.5	12.7	13.17	1.00	0.51	1.66
6	70	13428	14.2	2.3	12.9	13.17	1.00	0.52	1.64
7	70	13435	14.2	2.2	13.0	13.17	1.00	0.52	1.62
8	70	13442	14.2	2.2	13.0	13.17	1.00	0.52	1.62
9	70	13449	14.2	2.1	13.0	13.17	1.00	0.52	1.62
10	70	13456	14.2	2.2	13.0	13.17	1.00	0.52	1.62
11	70	13463	14.2	2.2	12.9	13.17	1.00	0.52	1.63
12	70	13470	14.2	2.4	12.8	13.17	1.00	0.51	1.64
13	70	13477	14.2	2.6	12.6	13.17	1.00	0.50	1.67
14	70	13484	14.2	2.9	12.3	13.17	1.00	0.49	1.71
15	70	13491	14.2	3.6	11.6	13.17	1.00	0.46	1.82
16	70	13498	14.2	3.8	11.4	13.17	1.00	0.45	1.85
17	70	13505	14.2	2.9	12.3	13.17	1.00	0.49	1.72
18	70	13512	14.2	2.6	12.6	13.17	1.00	0.51	1.67
19	70	13519	14.2	2.3	12.8	13.17	1.00	0.51	1.64
20	70	13526	14.2	2.2	13.0	13.17	1.00	0.52	1.62
21	70	13533	14.2	2.1	13.1	13.17	1.00	0.52	1.61
22	70	13540	14.2	2.0	13.1	13.17	1.00	0.53	1.60
23	70	13547	14.2	2.0	13.2	13.17	1.00	0.53	1.60
24	70	13554	14.2	2.1	13.1	13.17	1.00	0.53	1.61
25	70	13561	14.2	2.1	13.1	13.17	1.00	0.52	1.61
26	70	13568	14.2	2.2	12.9	13.17	1.00	0.52	1.63
27	70	13575	14.2	2.5	12.7	13.17	1.00	0.51	1.66
28	70	13582	14.2	2.9	12.3	13.17	1.00	0.49	1.71
29	70	13589	14.2	4.4	10.8	13.17	1.00	0.43	1.95
30	70	13596	14.2	3.0	12.2	13.17	1.00	0.49	1.73
31	70	13603	14.2	2.4	12.8	13.17	1.00	0.51	1.65
32	70	13610	14.2	2.0	13.2	13.17	1.00	0.53	1.60
33	70	13617	14.2	1.8	13.4	13.17	1.00	0.54	1.57
34	70	13624	14.2	1.6	13.6	13.17	1.00	0.54	1.55
35	70	13631	14.2	1.4	13.8	13.17	1.00	0.55	1.53
36	70	13638	14.2	1.3	13.9	13.17	1.00	0.56	1.51
37	70	13645	14.2	1.1	14.0	13.17	1.00	0.56	1.50
38	70	13652	14.2	1.0	14.1	13.17	1.00	0.57	1.49
39	70	13659	14.2	1.0	14.2	13.17	1.00	0.57	1.49
40	70	13666	14.2	0.9	14.2	13.17	1.00	0.57	1.49
41	70	13673	14.2	0.8	14.2	13.17	1.00	0.57	1.49



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 134+00 to 136+00
Critical Station = 135+47
Head Above Ground Surface (ft)
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
Distance from Seepage Entrance Point (ft) (positive indicates landward)
Without Wells
With Wells
FSreq
Landside Toe
Bluff


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 134+00 to 136+00
Landside Toe
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
13400
13450
13500
13550
13600
Station
Without Wells
With Wells
FSreq

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	57	ft
h_o =	16.05	ft
TOL =	770.1	ft elevation
Landside =	753.0	ft elevation
Bottom Blanket =	727	ft elevation
blanket =	26.0	ft
z, Landside Toe =	70	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	15.30	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
8	70	13400	759.0	0.92	0.29	1.00
9	70	13495	759.0	0.86	0.28	1.00
10	70	13590	759.0	0.80	0.26	1.00
11	70	13645	759.0	0.77	0.25	1.00
12	70	13705	759.0	0.77	0.24	1.00
13	70	13770	759.0	0.76	0.24	1.00
14	70	13830	759.0	0.75	0.24	1.00
15	70	13890	759.0	0.75	0.24	1.00
16	70	13950	759.0	0.75	0.24	1.00
17	70	14010	759.0	0.75	0.24	1.00
18	70	14070	759.0	0.75	0.24	1.00
19	70	14130	759.0	0.75	0.24	1.00
20	70	14190	759.0	0.74	0.23	1.00
21	70	14250	759.0	0.70	0.22	1.00
22	70	14275	759.0	0.69	0.22	1.00
23	70	14300	759.0	0.74	0.24	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

<--Input H_w AVG after any changes are made to well parameters

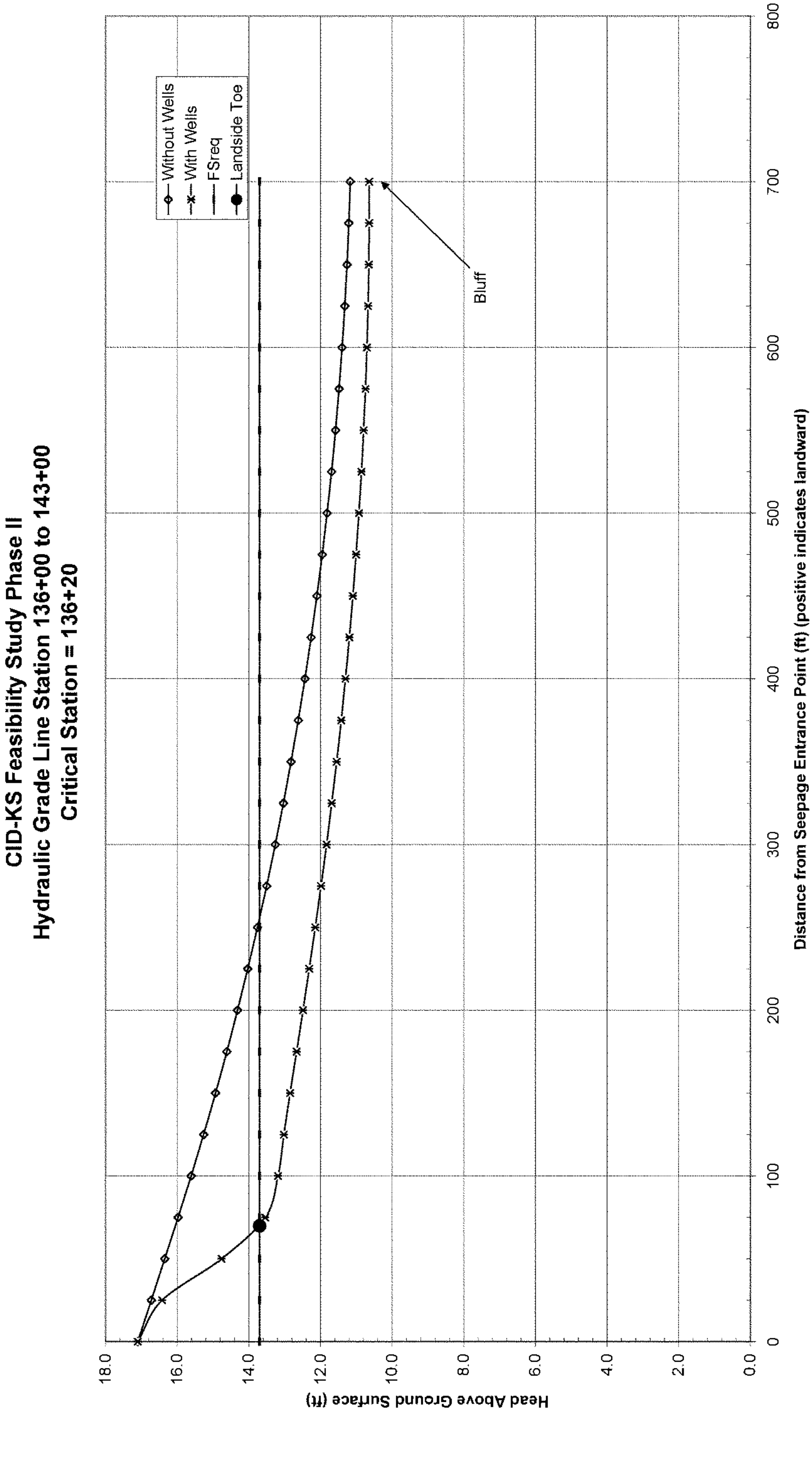
image well locations		
well	x'	y
8	-70	13400
9	-70	13495
10	-70	13590
11	-70	13645
12	-70	13705
13	-70	13770
14	-70	13830
15	-70	13890
16	-70	13950
17	-70	14010
18	-70	14070
19	-70	14130
20	-70	14190
21	-70	14250
22	-70	14275
23	-70	14300
0	0	0
0	0	0
0	0	0
0	0	0

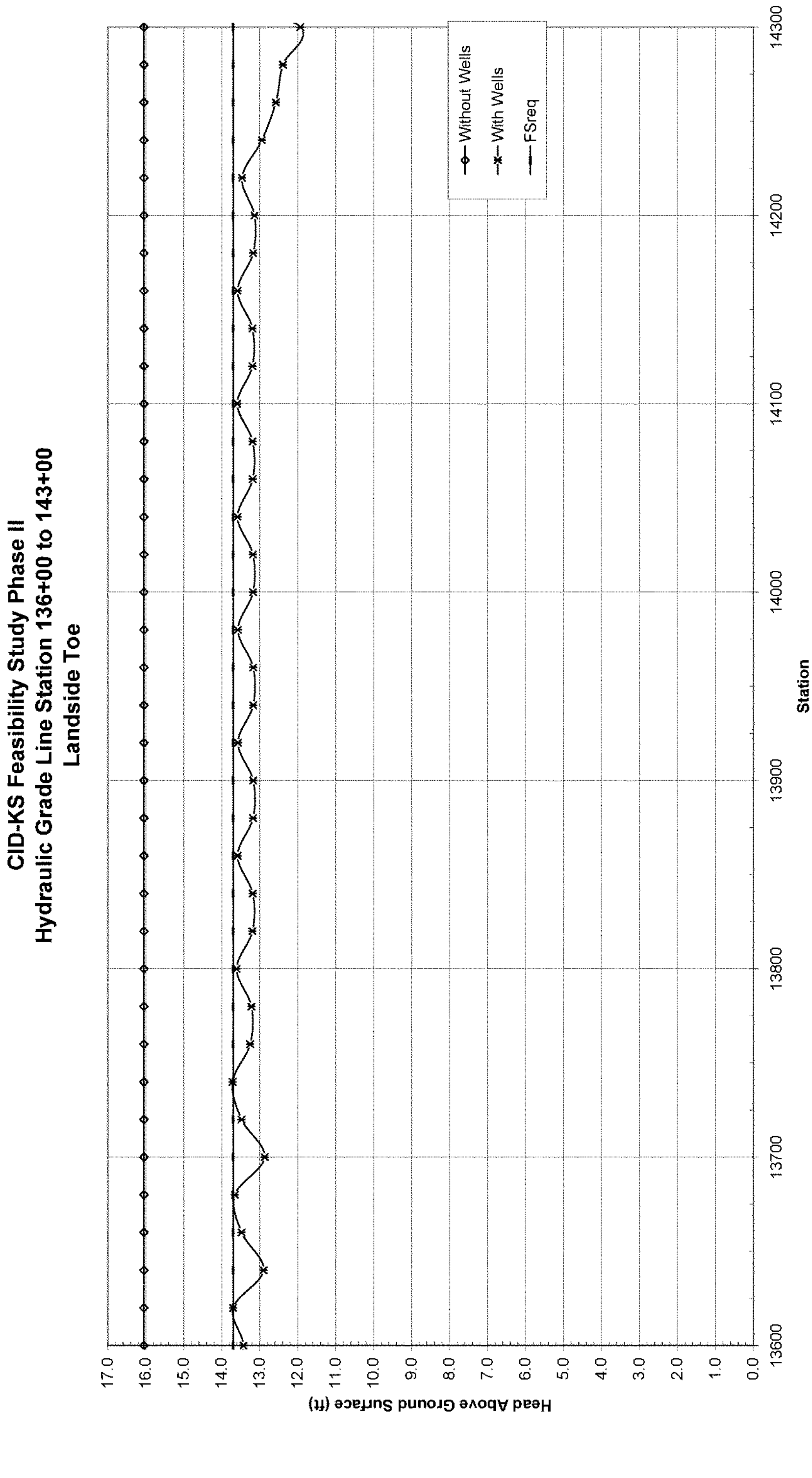
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_o (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	13620	17.1	0.0	17.1	13.70	1.00	0.66	1.28
2	25	13620	16.7	1.3	16.4	13.70	1.00	0.63	1.33
3	50	13620	16.3	2.6	14.8	13.70	1.00	0.57	1.48
4	75	13620	16.0	3.4	13.5	13.70	1.00	0.52	1.62
5	100	13620	15.6	3.4	13.2	13.70	1.00	0.51	1.66
6	125	13620	15.3	3.2	13.0	13.70	1.00	0.50	1.68
7	150	13620	14.9	3.1	12.8	13.70	1.00	0.49	1.71
8	175	13620	14.6	2.9	12.7	13.70	1.00	0.49	1.73
9	200	13620	14.3	2.8	12.5	13.70	1.00	0.48	1.76
10	225	13620	14.0	2.7	12.3	13.70	1.00	0.47	1.78
11	250	13620	13.8	2.6	12.1	13.70	1.00	0.47	1.80
12	275	13620	13.5	2.5	12.0	13.70	1.00	0.46	1.83
13	300	13620	13.3	2.4	11.8	13.70	1.00	0.46	1.85
14	325	13620	13.0	2.4	11.7	13.70	1.00	0.45	1.88
15	350	13620	12.8	2.3	11.5	13.70	1.00	0.44	1.90
16	375	13620	12.6	2.2	11.4	13.70	1.00	0.44	1.92
17	400	13620	12.4	2.1	11.3	13.70	1.00	0.43	1.94
18	425	13620	12.3	2.1	11.2	13.70	1.00	0.43	1.96
19	450	13620	12.1	2.0	11.1	13.70	1.00	0.43	1.98
20	475	13620	12.0	1.9	11.0	13.70	1.00	0.42	1.99
21	500	13620	11.8	1.9	10.9	13.70	1.00	0.42	2.01
22	525	13620	11.7	1.8	10.9	13.70	1.00	0.42	2.02
23	550	13620	11.6	1.8	10.8	13.70	1.00	0.42	2.03
24	575	13620	11.5	1.7	10.7	13.70	1.00	0.41	2.04
25	600	13620	11.4	1.7	10.7	13.70	1.00	0.41	2.05
26	625	13620	11.3	1.6	10.7	13.70	1.00	0.41	2.05
27	650	13620	11.3	1.6	10.7	13.70	1.00	0.41	2.06
28	675	13620	11.2	1.6	10.6	13.70	1.00	0.41	2.06
29	700	13620	11.2	1.5	10.6	13.70	1.00	0.41	2.06

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_o (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	70	13580	16.0	3.4	13.6	13.70	1.00	0.52	1.61
2	70	13600	16.0	3.6	13.4	13.70	1.00	0.52	1.63
3	70	13620	16.0	3.3	13.7	13.70	1.00	0.53	1.60
4	70	13640	16.0	4.2	12.9	13.70	1.00	0.50	1.70
5	70	13660	16.0	3.5	13.5	13.70	1.00	0.52	1.63
6	70	13680	16.0	3.4	13.7	13.70	1.00	0.53	1.60
7	70	13700	16.0	4.2	12.9	13.70	1.00	0.50	1.70
8	70	13720	16.0	3.6	13.5	13.70	1.00	0.52	1.63
9	70	13740	16.0	3.3	13.7	13.70	1.00	0.53	1.60
10	70	13760	16.0	3.8	13.3	13.70	1.00	0.51	1.65
11	70	13780	16.0	3.8	13.2	13.70	1.00	0.51	1.66
12	70	13800	16.0	3.4	13.6	13.70	1.00	0.52	1.61
13	70	13820	16.0	3.9	13.2	13.70	1.00	0.51	1.66
14	70	13840	16.0	3.9	13.2	13.70	1.00	0.51	1.66
15	70	13860	16.0	3.5	13.6	13.70	1.00	0.52	1.61
16	70	13880	16.0	3.9	13.2	13.70	1.00	0.51	1.66
17	70	13900	16.0	3.9	13.2	13.70	1.00	0.51	1.66
18	70	13920	16.0	3.5	13.6	13.70	1.00	0.52	1.61
19	70	13940	16.0	3.9	13.2	13.70	1.00	0.51	1.66
20	70	13960	16.0	3.9	13.2	13.70	1.00	0.51	1.66
21	70	13980	16.0	3.5	13.6	13.70	1.00	0.52	1.61
22	70	14000	16.0	3.9	13.2	13.70	1.00	0.51	1.66
23	70	14020	16.0	3.9	13.2	13.70	1.00	0.51	1.66
24	70	14040	16.0	3.5	13.6	13.70	1.00	0.52	1.61
25	70	14060	16.0	3.9	13.2	13.70	1.00	0.51	1.66
26	70	14080	16.0	3.9	13.2	13.70	1.00	0.51	1.66
27	70	14100	16.0	3.5	13.6	13.70	1.00	0.52	1.61
28	70	14120	16.0	3.9	13.2	13.70	1.00	0.51	1.66
29	70	14140	16.0	3.9	13.2	13.70	1.00	0.51	1.66
30	70	14160	16.0	3.5	13.6	13.70	1.00	0.52	1.61
31	70	14180	16.0	3.9	13.2	13.70	1.00	0.51	1.66
32	70	14200	16.0	3.9	13.1	13.70	1.00	0.51	1.67
33	70	14220	16.0	3.6	13.5	13.70	1.00	0.52	1.63
34	70	14240	16.0	4.1	12.9	13.70	1.00	0.50	1.69
35	70	14260	16.0	4.5	12.6	13.70	1.00	0.48	1.74
36	70	14280	16.0	4.7	12.4	13.70	1.00	0.48	1.77
37	70	14300	16.0	5.1	11.9	13.70	1.00	0.46	1.84
38	70	14320	16.0	2.8	14.2	13.70	1.00	0.55	1.54
39	70	14340	16.0	2.1	15.0	13.70	1.00	0.56	1.47
40	70	14360	16.0	1.6	15.4	13.70	1.00	0.59	1.42
41	70	14380	16.0	1.3	15.7	13.70	1.00	0.61	1.39


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 136+00 to 143+00
Critical Station = 136+20
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
Without Wells
With Wells
FSreq
Landside Toe
Bluff
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 136+00 to 143+00
Landside Toe
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
13600
13700
13800
13900
14000
14100
14200
14300
Station
Without Wells
With Wells
FSreq

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	55	ft
h_0 =	17.16	ft
TOL =	770.5	ft elevation
Landside =	752.0	ft elevation
Bottom Blanket =	725	ft elevation
blanket =	23.0	ft
z, Landside Toe =	70	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	25.34	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
20	70	14190	759.0	0.76	0.24	1.00
21	70	14250	759.0	0.63	0.20	1.00
22	70	14275	759.0	0.59	0.19	1.00
23	70	14300	759.0	0.57	0.18	1.00
24	70	14325	752.0	0.97	0.31	1.00
25	70	14350	752.0	1.01	0.32	1.00
26	70	14400	752.0	1.08	0.34	1.01
27	70	14450	752.0	1.10	0.35	1.01
28	70	14500	752.0	1.12	0.36	1.01
29	70	14550	752.0	1.12	0.36	1.01
30	70	14600	752.0	1.13	0.36	1.01
31	70	14650	752.0	1.13	0.36	1.01
32	70	14700	752.0	1.13	0.36	1.01
33	70	14750	752.0	1.13	0.36	1.01
34	70	14800	752.0	1.14	0.36	1.01
35	70	14850	752.0	1.14	0.36	1.01
36	70	14900	752.0	1.14	0.38	1.01
37	70	14950	752.0	1.10	0.35	1.01
38	70	14975	752.0	1.10	0.35	1.01
39	70	15000	752.0	1.19	0.38	1.01
						1.00

<---Input H_w AVG after any changes are made to well parameters

image well locations		
well	x'	y'
20	-70	14190
21	-70	14250
22	-70	14275
23	-70	14300
24	-70	14325
25	-70	14350
26	-70	14400
27	-70	14450
28	-70	14500
29	-70	14550
30	-70	14600
31	-70	14650
32	-70	14700
33	-70	14750
34	-70	14800
35	-70	14850
36	-70	14900
37	-70	14950
38	-70	14975
39	-70	15000

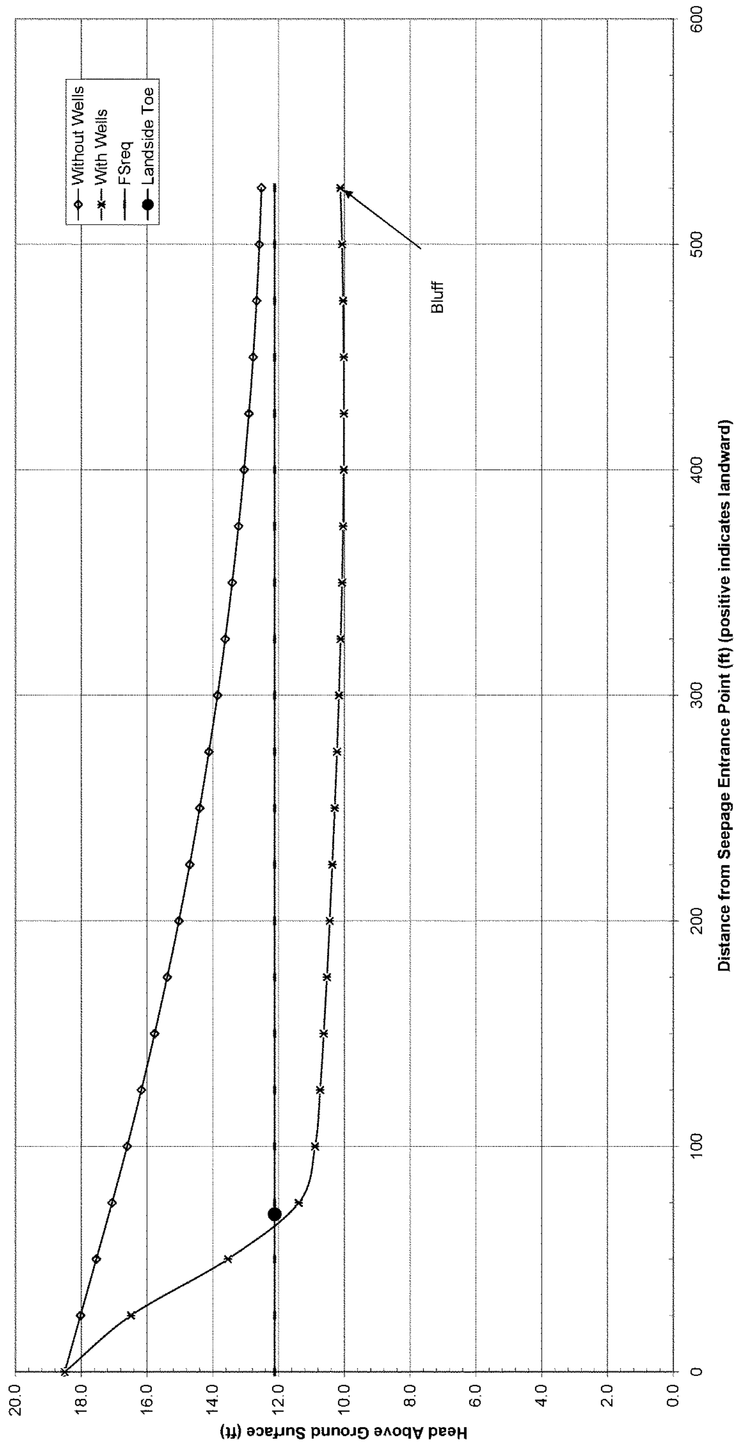
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	14420	18.5	0.0	18.5	12.12	1.00	0.80	1.05
2	25	14420	18.0	2.5	16.5	12.12	1.00	0.72	1.18
3	50	14420	17.5	5.0	13.5	12.12	1.00	0.59	1.43
4	75	14420	17.1	6.7	11.4	12.12	1.00	0.50	1.70
5	100	14420	16.6	6.7	10.9	12.12	1.00	0.47	1.78
6	125	14420	16.2	6.4	10.7	12.12	1.00	0.47	1.81
7	150	14420	15.8	6.1	10.6	12.12	1.00	0.46	1.82
8	175	14420	15.4	5.9	10.5	12.12	1.00	0.46	1.84
9	200	14420	15.0	5.6	10.4	12.12	1.00	0.45	1.86
10	225	14420	14.7	5.3	10.4	12.12	1.00	0.45	1.87
11	250	14420	14.4	5.1	10.3	12.12	1.00	0.45	1.88
12	275	14420	14.1	4.9	10.2	12.12	1.00	0.44	1.90
13	300	14420	13.9	4.7	10.2	12.12	1.00	0.44	1.91
14	325	14420	13.6	4.5	10.1	12.12	1.00	0.44	1.92
15	350	14420	13.4	4.3	10.1	12.12	1.00	0.44	1.93
16	375	14420	13.2	4.2	10.0	12.12	1.00	0.44	1.93
17	400	14420	13.0	4.0	10.0	12.12	1.00	0.44	1.93
18	425	14420	12.9	3.9	10.0	12.12	1.00	0.44	1.94
19	450	14420	12.8	3.8	10.0	12.12	1.00	0.44	1.93
20	475	14420	12.7	3.6	10.0	12.12	1.00	0.44	1.93
21	500	14420	12.6	3.5	10.1	12.12	1.00	0.44	1.93
22	525	14420	12.5	3.4	10.1	12.12	1.00	0.44	1.92

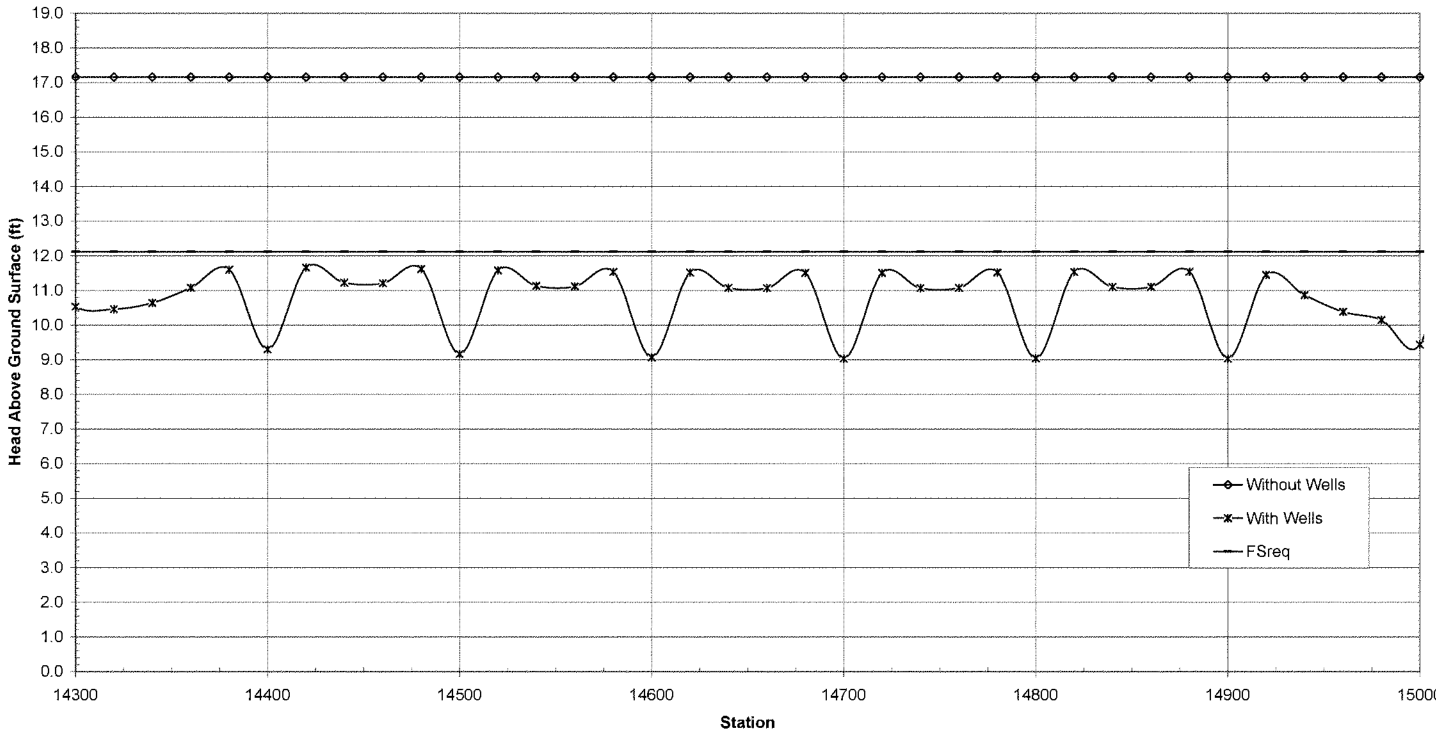
Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	70	14280	17.2	6.4	11.7	12.12	1.00	0.51	1.65
2	70	14300	17.2	7.6	10.5	12.12	1.00	0.46	1.84
3	70	14320	17.2	7.7	10.5	12.12	1.00	0.45	1.85
4	70	14340	17.2	7.5	10.6	12.12	1.00	0.46	1.82
5	70	14360	17.2	7.1	11.1	12.12	1.00	0.48	1.75
6	70	14380	17.2	6.6	11.6	12.12	1.00	0.50	1.67
7	70	14400	17.2	8.9	9.3	12.12	1.00	0.40	2.09
8	70	14420	17.2	6.5	11.7	12.12	1.00	0.51	1.66
9	70	14440	17.2	6.9	11.2	12.12	1.00	0.49	1.73
10	70	14460	17.2	8.9	11.2	12.12	1.00	0.49	1.73
11	70	14480	17.2	6.5	11.6	12.12	1.00	0.51	1.67
12	70	14500	17.2	9.0	9.2	12.12	1.00	0.40	2.12
13	70	14520	17.2	6.6	11.6	12.12	1.00	0.50	1.67
14	70	14540	17.2	7.0	11.1	12.12	1.00	0.48	1.74
15	70	14560	17.2	7.0	11.1	12.12	1.00	0.48	1.74
16	70	14580	17.2	6.6	11.5	12.12	1.00	0.50	1.68
17	70	14600	17.2	9.1	9.1	12.12	1.00	0.39	2.14
18	70	14620	17.2	6.6	11.5	12.12	1.00	0.50	1.68
19	70	14640	17.2	7.1	11.1	12.12	1.00	0.48	1.75
20	70	14660	17.2	7.1	11.1	12.12	1.00	0.48	1.75
21	70	14680	17.2	6.7	11.5	12.12	1.00	0.50	1.69
22	70	14700	17.2	9.1	9.0	12.12	1.00	0.39	2.15
23	70	14720	17.2	6.7	11.5	12.12	1.00	0.50	1.69
24	70	14740	17.2	7.1	11.1	12.12	1.00	0.48	1.75
25	70	14760	17.2	7.1	11.1	12.12	1.00	0.48	1.75
26	70	14780	17.2	6.6	11.5	12.12	1.00	0.50	1.68
27	70	14800	17.2	9.1	9.0	12.12	1.00	0.39	2.14
28	70	14820	17.2	6.6	11.5	12.12	1.00	0.50	1.68
29	70	14840	17.2	7.1	11.1	12.12	1.00	0.48	1.75
30	70	14860	17.2	7.1	11.1	12.12	1.00	0.48	1.75
31	70	14880	17.2	6.6	11.5	12.12	1.00	0.50	1.68
32	70	14900	17.2	9.1	9.0	12.12	1.00	0.39	2.15
33	70	14920	17.2	6.7	11.4	12.12	1.00	0.50	1.69
34	70	14940	17.2	7.3	10.9	12.12	1.00	0.47	1.78
35	70	14960	17.2	7.8	10.4	12.12	1.00	0.45	1.87
36	70	14980	17.2	8.0	10.2	12.12	1.00	0.44	1.91
37	70	15000	17.2	8.7	9.4	12.12	1.00	0.41	2.05
38	70	15020	17.2	4.9	13.3	12.12	1.00	0.58	1.46
39	70	15040	17.2	3.6	14.5	12.12	1.00	0.63	1.33
40	70	15060	17.2	2.8	15.3	12.12	1.00	0.67	1.27
41	70	15080	17.2	2.3	15.9	12.12	1.00	0.69	1.22

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 143+00 to 150+00
Critical Station = 144+20



CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 143+00 to 150+00
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	55	ft
h_e =	18.25	ft
TOL =	770.9	ft elevation
Landside =	752.0	ft elevation
Bottom Blanket =	725	ft elevation
blanket =	23.0	ft
z, Landside Toe =	70	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	20.15	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
36	70	14900	752.0	1.29	0.41	1.01
37	70	14950	752.0	1.11	0.35	1.01
38	70	14975	752.0	1.06	0.34	1.00
39	70	15000	752.0	1.06	0.34	1.00
40	70	15050	752.0	1.06	0.34	1.00
41	70	15075	752.0	1.06	0.34	1.00
42	70	15125	752.0	1.09	0.35	1.01
43	70	15165	752.0	1.09	0.35	1.01
44	70	15205	752.0	1.10	0.35	1.01
45	70	15245	752.0	1.12	0.36	1.01
46	70	15285	752.0	1.15	0.37	1.01
47	70	15310	752.0	1.22	0.39	1.01
48	450	15000	752.0	0.91	0.29	1.00
49	450	15150	752.0	0.88	0.28	1.00
50	450	15300	752.0	0.93	0.29	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00

image well locations		
well	x'	y'
36	-70	14900
37	-70	14950
38	-70	14975
39	-70	15000
40	-70	15050
41	-70	15075
42	-70	15125
43	-70	15165
44	-70	15205
45	-70	15245
46	-70	15285
47	-70	15310
48	-450	15000
49	-450	15150
50	-450	15300
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

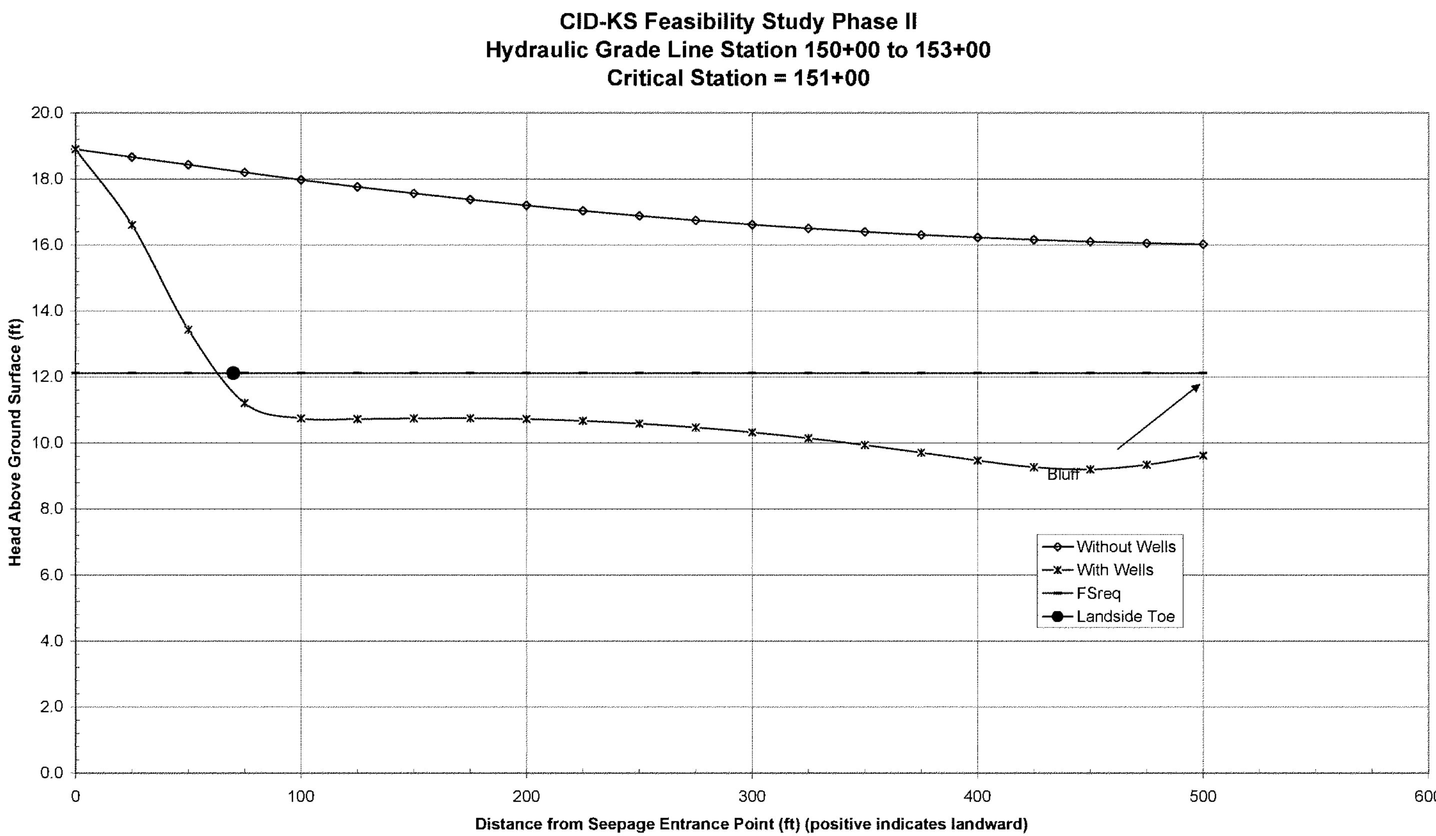
1.00 **<---Input H_w AVG after any changes are made to well parameters**

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

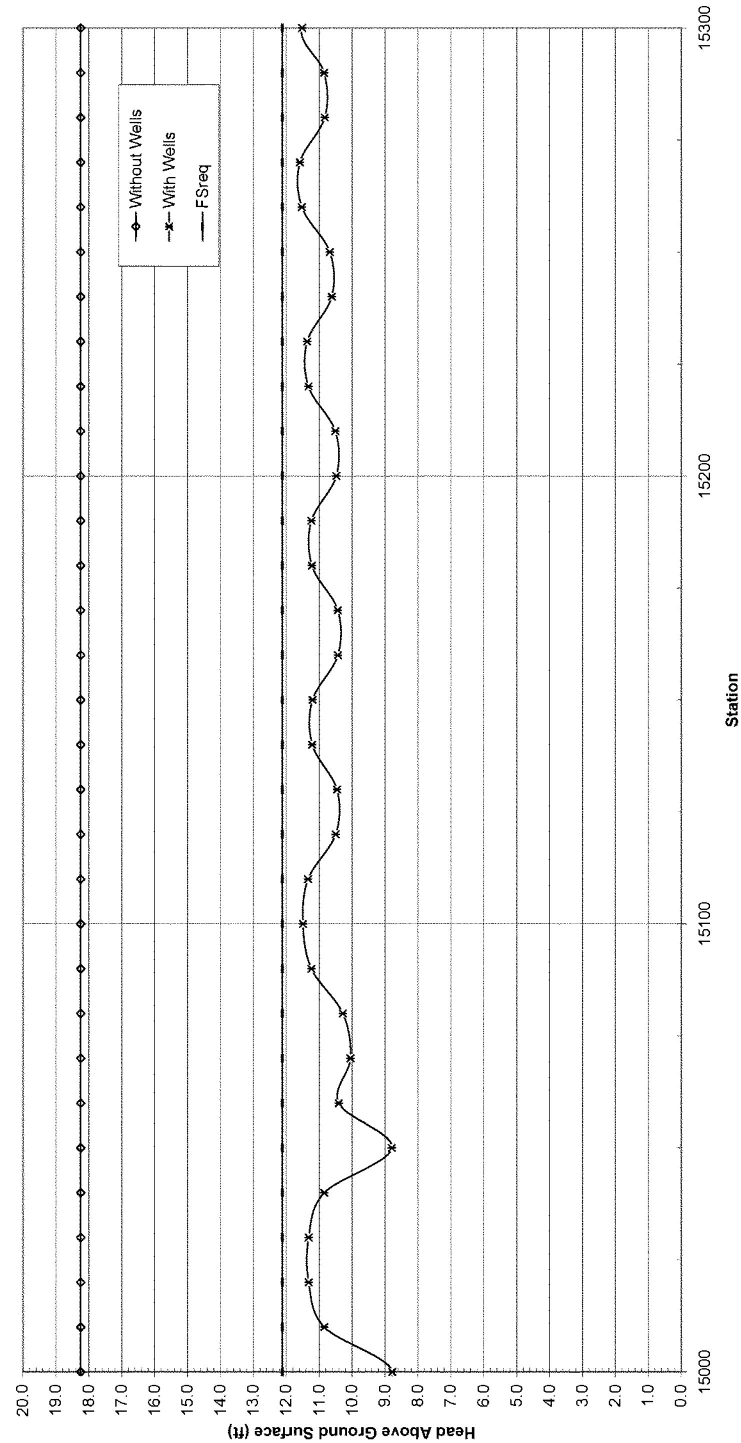
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	15100	18.9	0.0	18.9	12.12	1.00	0.82	1.03
2	25	15100	18.7	3.1	16.6	12.12	1.00	0.72	1.17
3	50	15100	18.4	6.0	13.4	12.12	1.00	0.58	1.44
4	75	15100	18.2	8.0	11.2	12.12	1.00	0.49	1.73
5	100	15100	18.0	8.2	10.7	12.12	1.00	0.47	1.80
6	125	15100	17.8	8.0	10.7	12.12	1.00	0.47	1.81
7	150	15100	17.6	7.8	10.7	12.12	1.00	0.47	1.80
8	175	15100	17.4	7.6	10.8	12.12	1.00	0.47	1.80
9	200	15100	17.2	7.5	10.7	12.12	1.00	0.47	1.81
10	225	15100	17.0	7.4	10.7	12.12	1.00	0.46	1.82
11	250	15100	16.9	7.3	10.6	12.12	1.00	0.46	1.83
12	275	15100	16.7	7.3	10.5	12.12	1.00	0.46	1.85
13	300	15100	16.6	7.3	10.3	12.12	1.00	0.45	1.88
14	325	15100	16.5	7.4	10.1	12.12	1.00	0.44	1.91
15	350	15100	16.4	7.5	9.9	12.12	1.00	0.43	1.95
16	375	15100	16.3	7.6	9.7	12.12	1.00	0.42	2.00
17	400	15100	16.2	7.8	9.5	12.12	1.00	0.41	2.05
18	425	15100	16.2	7.9	9.3	12.12	1.00	0.40	2.09
19	450	15100	16.1	7.9	9.2	12.12	1.00	0.40	2.11
20	475	15100	16.1	7.7	9.3	12.12	1.00	0.41	2.08
21	500	15100	16.0	7.4	9.6	12.12	1.00	0.42	2.01

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	70	14990	18.2	8.9	10.4	12.12	1.00	0.45	1.87
2	70	15000	18.2	10.5	8.8	12.12	1.00	0.38	2.21
3	70	15010	18.2	8.4	10.8	12.12	1.00	0.47	1.79
4	70	15020	18.2	7.9	11.3	12.12	1.00	0.49	1.71
5	70	15030	18.2	7.9	11.3	12.12	1.00	0.49	1.71
6	70	15040	18.2	8.4	10.8	12.12	1.00	0.47	1.79
7	70	15050	18.2	10.4	8.8	12.12	1.00	0.38	2.20
8	70	15060	18.2	8.8	10.4	12.12	1.00	0.45	1.86
9	70	15070	18.2	9.2	10.0	12.12	1.00	0.44	1.93
10	70	15080	18.2	9.0	10.3	12.12	1.00	0.45	1.89
11	70	15090	18.2	8.0	11.2	12.12	1.00	0.49	1.73
12	70	15100	18.2	7.8	11.5	12.12	1.00	0.50	1.69
13	70	15110	18.2	7.9	11.3	12.12	1.00	0.49	1.71
14	70	15120	18.2	8.7	10.5	12.12	1.00	0.46	1.85
15	70	15130	18.2	8.8	10.5	12.12	1.00	0.45	1.85
16	70	15140	18.2	8.0	11.2	12.12	1.00	0.49	1.73
17	70	15150	18.2	8.0	11.2	12.12	1.00	0.49	1.73
18	70	15160	18.2	8.8	10.4	12.12	1.00	0.45	1.86
19	70	15170	18.2	8.8	10.4	12.12	1.00	0.45	1.86
20	70	15180	18.2	8.0	11.2	12.12	1.00	0.49	1.73
21	70	15190	18.2	8.0	11.2	12.12	1.00	0.49	1.73
22	70	15200	18.2	8.8	10.5	12.12	1.00	0.46	1.85
23	70	15210	18.2	8.7	10.5	12.12	1.00	0.46	1.84
24	70	15220	18.2	7.9	11.3	12.12	1.00	0.49	1.71
25	70	15230	18.2	7.9	11.4	12.12	1.00	0.49	1.71
26	70	15240	18.2	8.6	10.6	12.12	1.00	0.46	1.83
27	70	15250	18.2	8.6	10.7	12.12	1.00	0.46	1.82
28	70	15260	18.2	7.7	11.5	12.12	1.00	0.50	1.68
29	70	15270	18.2	7.7	11.6	12.12	1.00	0.50	1.67
30	70	15280	18.2	8.4	10.8	12.12	1.00	0.47	1.79
31	70	15290	18.2	8.4	10.9	12.12	1.00	0.47	1.79
32	70	15300	18.2	7.7	11.5	12.12	1.00	0.50	1.68
33	70	15310	18.2	9.2	10.0	12.12	1.00	0.44	1.94
34	70	15320	18.2	6.5	12.8	12.12	1.00	0.56	1.52
35	70	15330	18.2	5.4	13.9	12.12	1.00	0.60	1.40
36	70	15340	18.2	4.7	14.6	12.12	1.00	0.63	1.33
37	70	15350	18.2	4.1	15.1	12.12	1.00	0.66	1.26
38	70	15360	18.2	3.7	15.6	12.12	1.00	0.68	1.24
39	70	15370	18.2	3.3	15.9	12.12	1.00	0.69	1.22
40	70	15380	18.2	3.0	16.2	12.12	1.00	0.71	1.19
41	70	15390	18.2	2.8	16.5	12.12	1.00	0.72	1.18


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 150+00 to 153+00
Critical Station = 151+00
Head Above Ground Surface (ft)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
Distance from Seepage Entrance Point (ft) (positive indicates landward)
Bluff
Without Wells
With Wells
FSreq
Landside Toe

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 150+00 to 153+00
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	55	ft
h_0 =	17.57	ft
TOL =	771.2	ft elevation
Landside =	753.0	ft elevation
Bottom Blanket =	725	ft elevation
blanket =	23.0	ft
z, Landside Toe =	70	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	25.41	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
45	70	15245	752.0	1.33	0.42	1.01
46	70	15285	752.0	1.20	0.38	1.01
47	70	15310	752.0	1.19	0.38	1.01
50	450	15300	752.0	0.99	0.32	1.00
51	70	15370	753.0	1.15	0.37	1.01
52	70	15420	753.0	1.14	0.36	1.01
53	70	15470	753.0	1.13	0.36	1.01
54	70	15520	753.0	1.12	0.36	1.01
55	70	15570	753.0	1.12	0.36	1.01
56	70	15620	753.0	1.12	0.36	1.01
57	70	15670	753.0	1.13	0.36	1.01
58	70	15720	753.0	1.13	0.36	1.01
59	70	15770	753.0	1.14	0.36	1.01
60	70	15820	753.0	1.16	0.37	1.01
61	70	15870	753.0	1.18	0.38	1.01
62	70	15900	753.0	1.24	0.40	1.01
63	400	15600	753.0	0.89	0.28	1.00
64	400	15900	753.0	0.96	0.31	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00 <---**Input H_w AVG after any changes are made to well parameters**

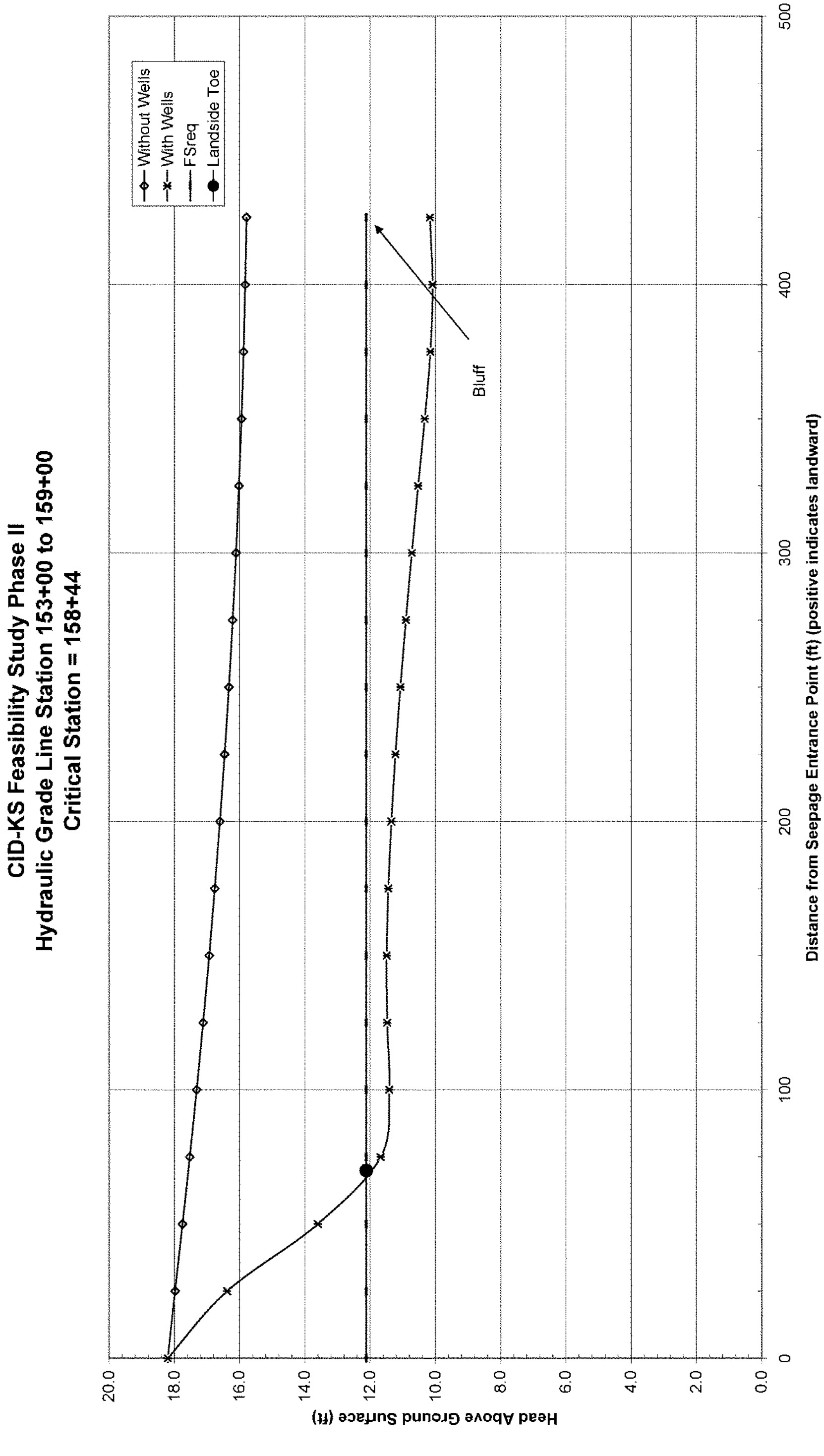
image well locations		
well	x'	y'
45	-70	15245
46	-70	15285
47	-70	15310
50	-450	15300
51	-70	15370
52	-70	15420
53	-70	15470
54	-70	15520
55	-70	15570
56	-70	15620
57	-70	15670
58	-70	15720
59	-70	15770
60	-70	15820
61	-70	15870
62	-70	15900
63	-400	15600
64	-400	15900
0	0	0
0	0	0

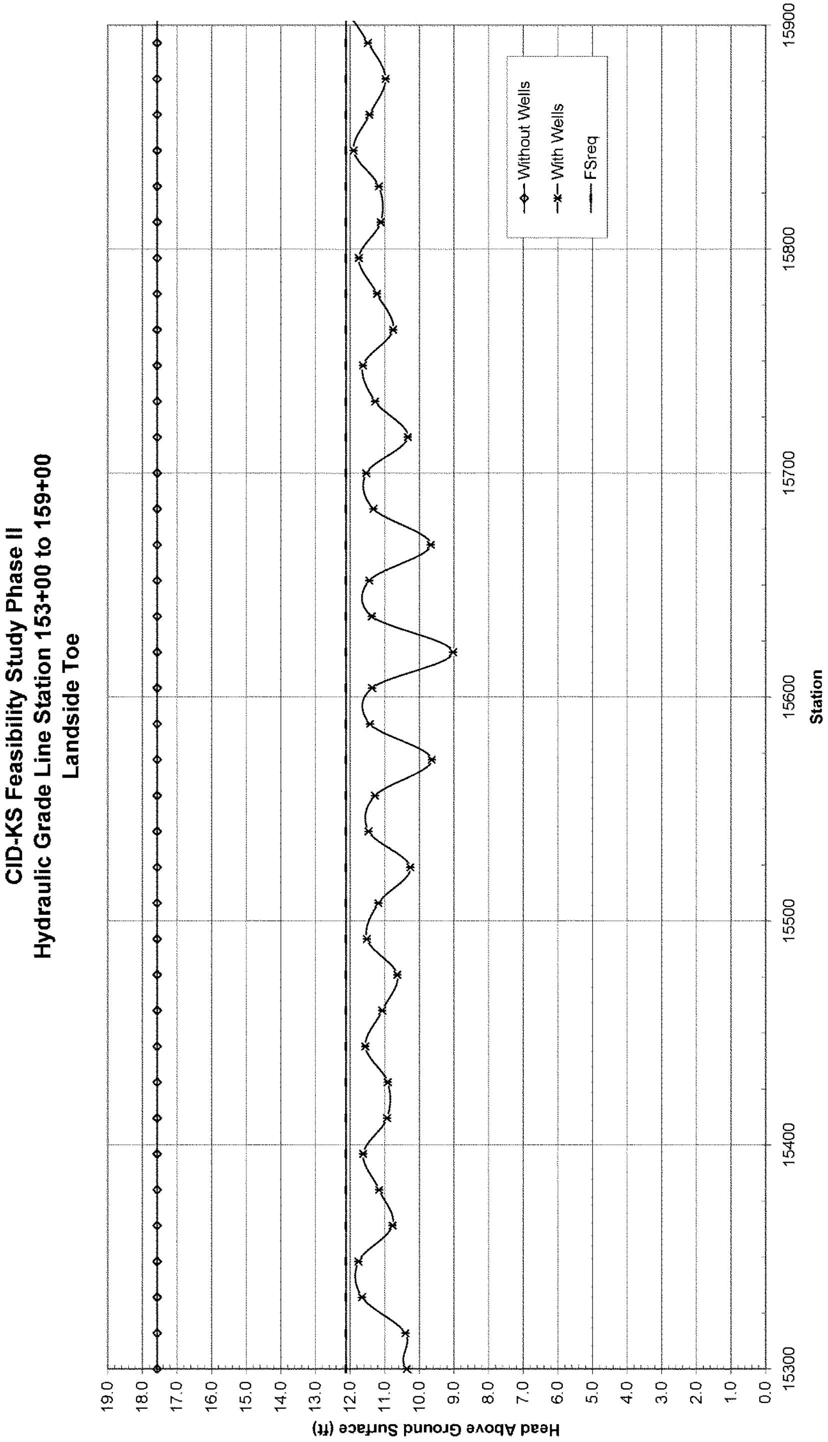
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	15844	18.2	0.0	18.2	12.12	1.00	0.79	1.07
2	25	15844	18.0	2.6	16.4	12.12	1.00	0.71	1.18
3	50	15844	17.8	5.1	13.6	12.12	1.00	0.59	1.43
4	75	15844	17.5	6.9	11.7	12.12	1.00	0.51	1.66
5	100	15844	17.3	6.9	11.4	12.12	1.00	0.50	1.70
6	125	15844	17.1	6.8	11.5	12.12	1.00	0.50	1.69
7	150	15844	16.9	6.4	11.5	12.12	1.00	0.50	1.69
8	175	15844	16.8	6.3	11.4	12.12	1.00	0.50	1.69
9	200	15844	16.6	6.3	11.4	12.12	1.00	0.49	1.71
10	225	15844	16.5	6.2	11.2	12.12	1.00	0.49	1.73
11	250	15844	16.3	6.3	11.1	12.12	1.00	0.48	1.75
12	275	15844	16.2	6.3	10.9	12.12	1.00	0.47	1.78
13	300	15844	16.1	6.4	10.7	12.12	1.00	0.47	1.81
14	325	15844	16.0	6.5	10.5	12.12	1.00	0.46	1.84
15	350	15844	15.9	6.6	10.3	12.12	1.00	0.45	1.88
16	375	15844	15.9	6.7	10.2	12.12	1.00	0.44	1.91
17	400	15844	15.8	6.7	10.1	12.12	1.00	0.44	1.92
18	425	15844	15.8	6.6	10.2	12.12	1.00	0.44	1.91

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	70	15284	17.6	10.0	8.6	12.12	1.00	0.37	2.25
2	70	15300	17.6	8.2	10.4	12.12	1.00	0.45	1.87
3	70	15316	17.6	8.2	10.4	12.12	1.00	0.45	1.86
4	70	15332	17.6	6.9	11.7	12.12	1.00	0.51	1.66
5	70	15348	17.6	6.8	11.8	12.12	1.00	0.51	1.65
6	70	15364	17.6	7.8	10.8	12.12	1.00	0.47	1.80
7	70	15380	17.6	7.4	11.2	12.12	1.00	0.49	1.74
8	70	15396	17.6	7.0	11.6	12.12	1.00	0.51	1.67
9	70	15412	17.6	7.6	10.9	12.12	1.00	0.48	1.77
10	70	15428	17.6	7.7	10.9	12.12	1.00	0.47	1.78
11	70	15444	17.6	7.0	11.6	12.12	1.00	0.50	1.68
12	70	15460	17.6	7.5	11.1	12.12	1.00	0.48	1.75
13	70	15476	17.6	7.9	10.6	12.12	1.00	0.46	1.82
14	70	15492	17.6	7.1	11.5	12.12	1.00	0.50	1.68
15	70	15508	17.6	7.4	11.2	12.12	1.00	0.49	1.73
16	70	15524	17.6	8.3	10.3	12.12	1.00	0.45	1.89
17	70	15540	17.6	7.1	11.5	12.12	1.00	0.50	1.69
18	70	15556	17.6	7.3	11.3	12.12	1.00	0.49	1.72
19	70	15572	17.6	8.9	9.6	12.12	1.00	0.42	2.01
20	70	15588	17.6	7.2	11.4	12.12	1.00	0.50	1.70
21	70	15604	17.6	7.2	11.4	12.12	1.00	0.49	1.71
22	70	15620	17.6	9.6	9.0	12.12	1.00	0.39	2.15
23	70	15636	17.6	7.2	11.4	12.12	1.00	0.49	1.70
24	70	15652	17.6	7.1	11.4	12.12	1.00	0.50	1.69
25	70	15668	17.6	8.9	9.7	12.12	1.00	0.42	2.01
26	70	15684	17.6	7.2	11.3	12.12	1.00	0.49	1.71
27	70	15700	17.6	7.0	11.5	12.12	1.00	0.50	1.68
28	70	15716	17.6	8.2	10.3	12.12	1.00	0.45	1.88
29	70	15732	17.6	7.3	11.3	12.12	1.00	0.49	1.72
30	70	15748	17.6	6.9	11.6	12.12	1.00	0.51	1.67
31	70	15764	17.6	7.8	10.8	12.12	1.00	0.47	1.80
32	70	15780	17.6	7.4	11.2	12.12	1.00	0.49	1.73
33	70	15796	17.6	6.8	11.7	12.12	1.00	0.51	1.65
34	70	15812	17.6	7.5	11.1	12.12	1.00	0.46	1.74
35	70	15828	17.6	7.4	11.2	12.12	1.00	0.49	1.74
36	70	15844	17.6	6.7	11.9	12.12	1.00	0.52	1.63
37	70	15860	17.6	7.1	11.4	12.12	1.00	0.50	1.69
38	70	15876	17.6	7.6	11.0	12.12	1.00	0.48	1.77
39	70	15892	17.6	7.1	11.5	12.12	1.00	0.50	1.69
40	70	15908	17.6	6.3	12.3	12.12	1.00	0.53	1.58
41	70	15924	17.6	4.7	13.9	12.12	1.00	0.00	1.40


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 153+00 to 159+00
Critical Station = 158+44
Without Wells
With Wells
FSreq
Landside Toe
Bluff
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 153+00 to 159+00
Landside Toe
Head Above Ground Surface (ft)
Station
Without Wells
With Wells
FSreq
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
15300
15400
15500
15600
15700
15800
15900

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	55	ft
h_0 =	17.09	ft
TOL =	771.5	ft elevation
Landside =	754.0	ft elevation
Bottom Blanket =	725	ft elevation
blanket =	21.0	ft
z, Landside Toe =	70	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	22.18	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
59	70	15770	753.0	1.34	0.43	1.01
60	70	15820	753.0	1.21	0.39	1.01
61	70	15870	753.0	1.12	0.36	1.01
62	70	15900	753.0	1.10	0.35	1.01
64	400	15900	753.0	0.94	0.30	1.00
65	70	15950	754.0	1.03	0.33	1.00
66	70	15990	754.0	1.01	0.32	1.00
67	70	16030	754.0	1.00	0.32	1.00
68	70	16070	754.0	0.99	0.32	1.00
69	70	16110	754.0	0.99	0.32	1.00
70	70	16150	754.0	1.00	0.32	1.00
71	70	16190	754.0	1.01	0.32	1.00
72	70	16230	754.0	1.03	0.33	1.00
73	70	16270	754.0	1.07	0.34	1.00
74	70	16300	754.0	1.14	0.36	1.01
75	300	16100	754.0	0.84	0.27	1.00
76	300	16300	754.0	0.91	0.29	1.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
						1.00

image well locations		
well	x'	y'
59	-70	15770
60	-70	15820
61	-70	15870
62	-70	15900
64	-400	15900
65	-70	15950
66	-70	15990
67	-70	16030
68	-70	16070
69	-70	16110
70	-70	16150
71	-70	16190
72	-70	16230
73	-70	16270
74	-70	16300
75	-300	16100
76	-300	16300
0	0	0
0	0	0
0	0	0

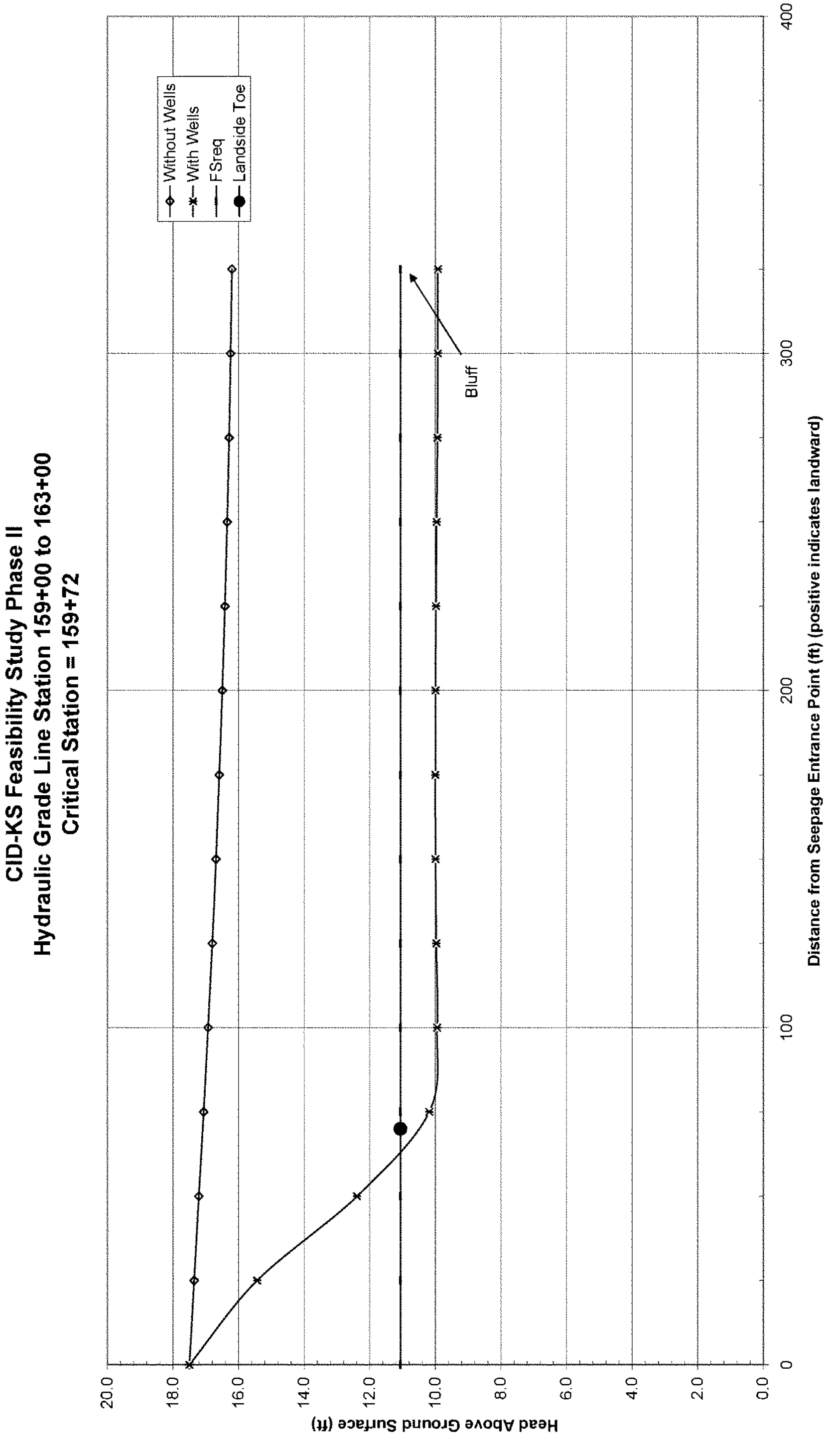
<---Input H_w AVG after any changes are made to well parameters

Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

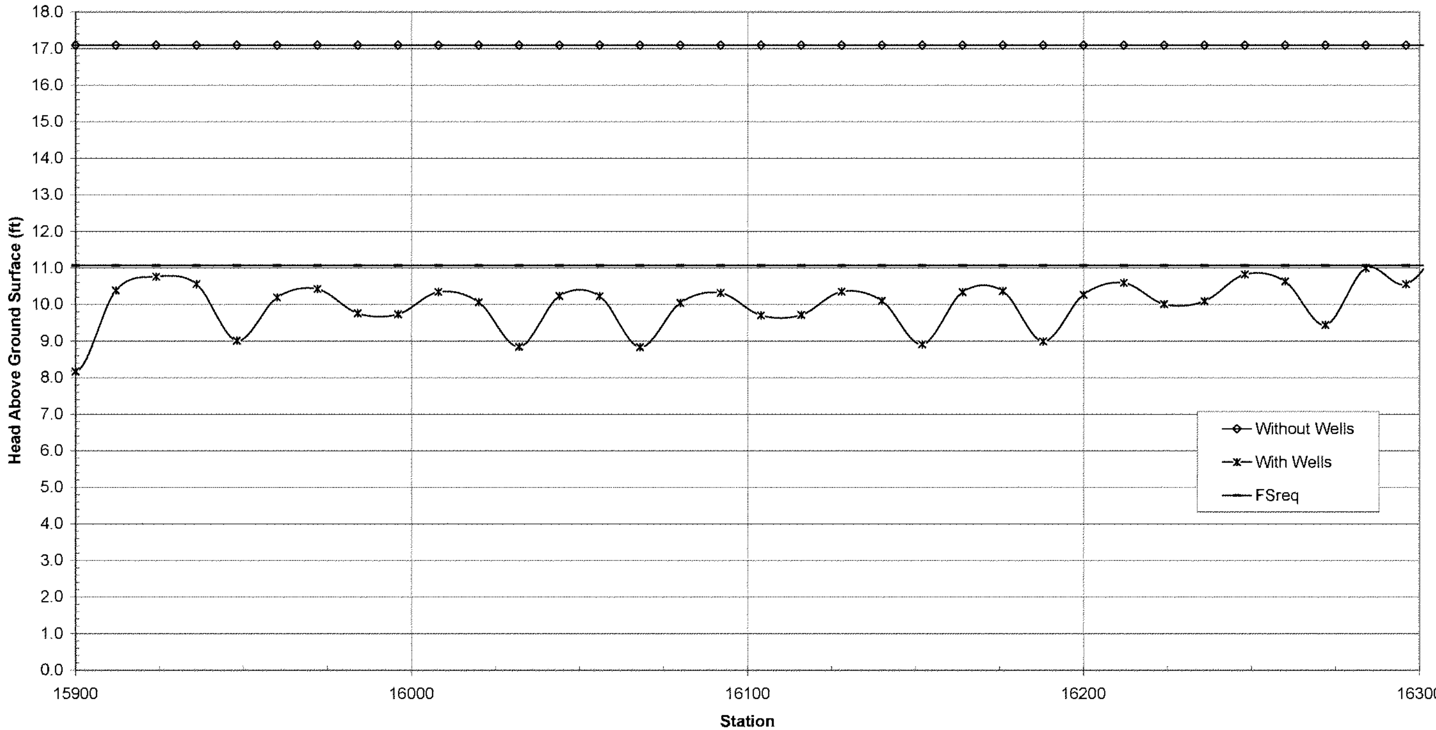
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	15972	17.5	0.0	17.5	11.06	1.00	0.83	1.01
2	25	15972	17.4	2.9	15.4	11.06	1.00	0.73	1.15
3	50	15972	17.2	5.8	12.4	11.06	1.00	0.59	1.43
4	75	15972	17.1	7.9	10.2	11.06	1.00	0.48	1.74
5	100	15972	16.9	8.0	9.9	11.06	1.00	0.47	1.78
6	125	15972	16.8	7.8	10.0	11.06	1.00	0.47	1.78
7	150	15972	16.7	7.7	10.0	11.06	1.00	0.48	1.77
8	175	15972	16.6	7.6	10.0	11.06	1.00	0.48	1.77
9	200	15972	16.5	7.5	10.0	11.06	1.00	0.48	1.77
10	225	15972	16.4	7.4	10.0	11.06	1.00	0.48	1.77
11	250	15972	16.3	7.4	10.0	11.06	1.00	0.47	1.78
12	275	15972	16.3	7.4	9.9	11.06	1.00	0.47	1.78
13	300	15972	16.2	7.3	9.9	11.06	1.00	0.47	1.78
14	325	15972	16.2	7.3	9.9	11.06	1.00	0.47	1.78

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	70	15888	17.1	8.0	10.1	11.06	1.00	0.48	1.76
2	70	15900	17.1	9.9	8.2	11.06	1.00	0.39	2.17
3	70	15912	17.1	7.7	10.4	11.06	1.00	0.49	1.70
4	70	15924	17.1	7.3	10.8	11.06	1.00	0.51	1.64
5	70	15936	17.1	7.5	10.6	11.06	1.00	0.50	1.68
6	70	15948	17.1	9.1	9.0	11.06	1.00	0.43	1.96
7	70	15960	17.1	7.9	10.2	11.06	1.00	0.49	1.74
8	70	15972	17.1	7.7	10.4	11.06	1.00	0.50	1.70
9	70	15984	17.1	8.3	9.8	11.06	1.00	0.46	1.81
10	70	15996	17.1	8.4	9.7	11.06	1.00	0.46	1.82
11	70	16008	17.1	7.7	10.3	11.06	1.00	0.49	1.71
12	70	16020	17.1	8.0	10.1	11.06	1.00	0.48	1.76
13	70	16032	17.1	9.3	8.8	11.06	1.00	0.42	2.00
14	70	16044	17.1	7.9	10.2	11.06	1.00	0.49	1.73
15	70	16056	17.1	7.9	10.2	11.06	1.00	0.49	1.73
16	70	16068	17.1	9.3	8.8	11.06	1.00	0.42	2.00
17	70	16080	17.1	8.1	10.0	11.06	1.00	0.48	1.76
18	70	16092	17.1	7.8	10.3	11.06	1.00	0.49	1.72
19	70	16104	17.1	8.4	9.7	11.06	1.00	0.46	1.82
20	70	16116	17.1	8.4	9.7	11.06	1.00	0.46	1.82
21	70	16128	17.1	7.7	10.4	11.06	1.00	0.49	1.71
22	70	16140	17.1	8.0	10.1	11.06	1.00	0.48	1.75
23	70	16152	17.1	9.2	8.9	11.06	1.00	0.42	1.99
24	70	16164	17.1	7.8	10.3	11.06	1.00	0.49	1.71
25	70	16176	17.1	7.7	10.4	11.06	1.00	0.49	1.71
26	70	16188	17.1	9.1	9.0	11.06	1.00	0.43	1.97
27	70	16200	17.1	7.8	10.3	11.06	1.00	0.49	1.72
28	70	16212	17.1	7.5	10.6	11.06	1.00	0.50	1.67
29	70	16224	17.1	8.1	10.0	11.06	1.00	0.48	1.77
30	70	16236	17.1	8.0	10.1	11.06	1.00	0.48	1.75
31	70	16248	17.1	7.3	10.8	11.06	1.00	0.52	1.64
32	70	16260	17.1	7.5	10.6	11.06	1.00	0.51	1.66
33	70	16272	17.1	8.7	9.4	11.06	1.00	0.45	1.87
34	70	16284	17.1	7.1	11.0	11.06	1.00	0.52	1.61
35	70	16296	17.1	7.5	10.6	11.06	1.00	0.50	1.68
36	70	16308	17.1	6.3	11.8	11.06	1.00	0.56	1.51
37	70	16320	17.1	5.1	13.0	11.06	1.00	0.62	1.36
38	70	16332	17.1	4.3	13.8	11.06	1.00	0.66	1.28
39	70	16344	17.1	3.7	14.4	11.06	1.00	0.68	1.23
40	70	16356	17.1	3.3	14.8	11.06	1.00	0.70	1.20
41	70	16368	17.1	3.0	15.1	11.06	1.00	0.72	1.17


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 159+00 to 163+00
Critical Station = 159+72
Without Wells
With Wells
FSreq
Landside Toe
Bluff
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400

CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 159+00 to 163+00
Landside Toe



RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	55	ft
h_o =	16.45	ft
TOL =	771.7	ft elevation
Landside =	755.0	ft elevation
Bottom Blanket =	725	ft elevation
blanket =	25.0	ft
z, Landside Toe =	70	ft
r_w =	1	ft

γ_{sat} =	115	pcf
i_c =	0.84	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	18.16	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
72	70	16230	754.0	1.28	0.41	1.01
73	70	16270	754.0	1.18	0.38	1.01
74	70	16300	754.0	1.20	0.38	1.01
75	300	16100	754.0	1.09	0.35	1.01
76	300	16300	754.0	1.01	0.32	1.00
77	70	16375	755.0	1.20	0.38	1.01
78	70	16450	755.0	1.23	0.39	1.01
79	70	16525	755.0	1.24	0.40	1.01
80	70	16600	755.0	1.25	0.40	1.01
81	70	16675	755.0	1.24	0.39	1.01
82	70	16725	755.0	1.26	0.40	1.01
83	70	16790	755.0	1.37	0.43	1.01
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00
				0.00	0.00	0.00

1.00 **<---Input H_w AVG after any changes are made to well parameters**

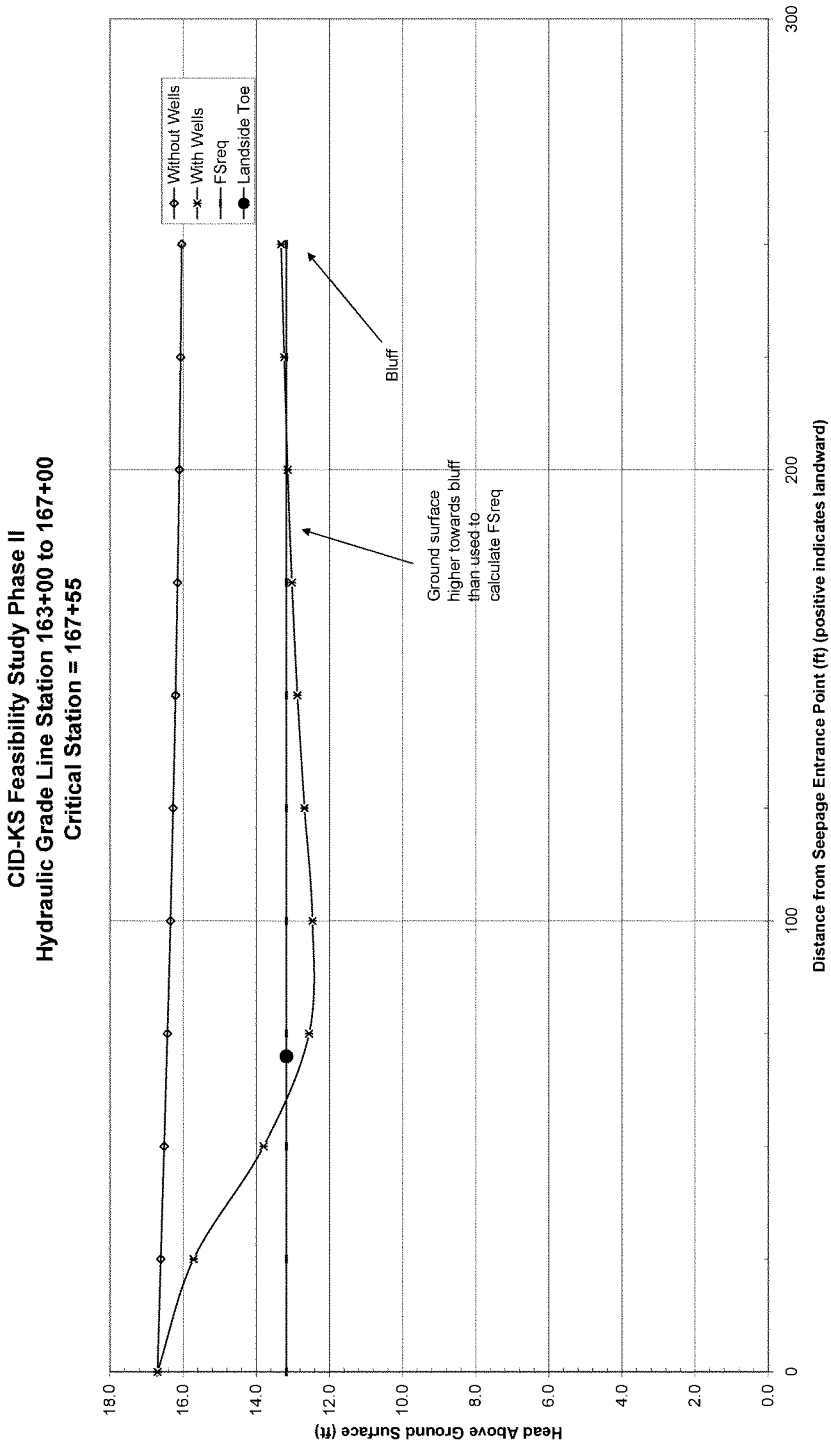
image well locations		
well	x'	y'
72	-70	16230
73	-70	16270
74	-70	16300
75	-300	16100
76	-300	16300
77	-70	16375
78	-70	16450
79	-70	16525
80	-70	16600
81	-70	16675
82	-70	16725
83	-70	16790
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0
0	0	0

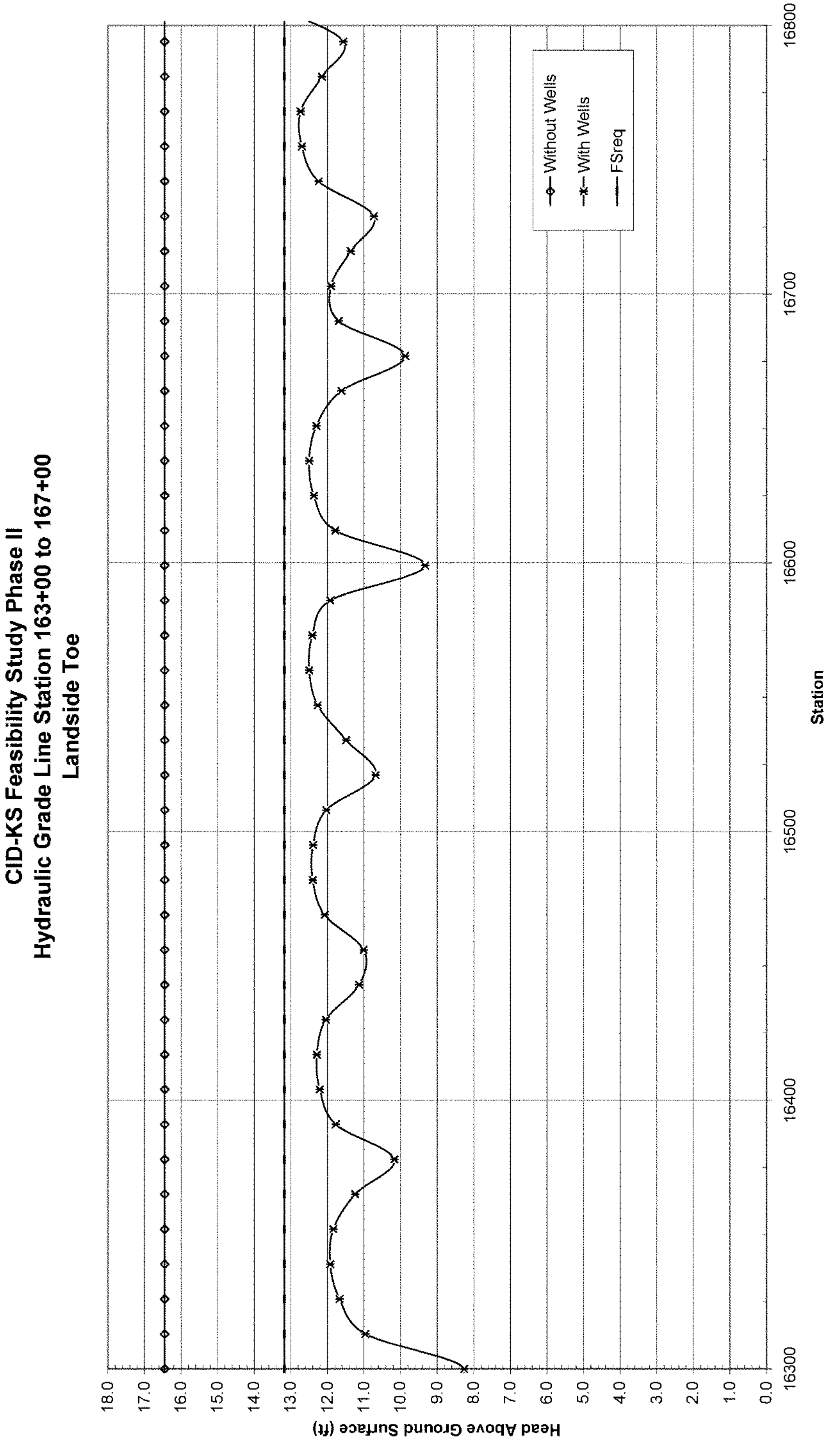
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	16755	16.7	0.0	16.7	13.17	1.00	0.67	1.26
2	25	16755	16.6	1.9	15.7	13.17	1.00	0.63	1.34
3	50	16755	16.5	3.7	13.8	13.17	1.00	0.55	1.53
4	75	16755	16.4	4.9	12.6	13.17	1.00	0.50	1.68
5	100	16755	16.3	4.9	12.5	13.17	1.00	0.50	1.69
6	125	16755	16.3	4.6	12.7	13.17	1.00	0.51	1.66
7	150	16755	16.2	4.3	12.9	13.17	1.00	0.52	1.64
8	175	16755	16.2	4.1	13.0	13.17	1.00	0.52	1.62
9	200	16755	16.1	4.0	13.1	13.17	1.00	0.53	1.60
10	225	16755	16.1	3.8	13.2	13.17	1.00	0.53	1.59
11	250	16755	16.0	3.7	13.3	13.17	1.00	0.53	1.58

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	70	16287	16.4	7.3	10.2	13.17	1.00	0.41	2.07
2	70	16300	16.4	9.2	8.3	13.17	1.00	0.33	2.55
3	70	16313	16.4	6.5	11.0	13.17	1.00	0.44	1.92
4	70	16326	16.4	5.8	11.7	13.17	1.00	0.47	1.81
5	70	16339	16.4	5.5	11.9	13.17	1.00	0.48	1.77
6	70	16352	16.4	5.6	11.8	13.17	1.00	0.47	1.78
7	70	16365	16.4	6.2	11.2	13.17	1.00	0.45	1.87
8	70	16378	16.4	7.3	10.2	13.17	1.00	0.41	2.07
9	70	16391	16.4	5.7	11.8	13.17	1.00	0.47	1.79
10	70	16404	16.4	5.2	12.2	13.17	1.00	0.49	1.73
11	70	16417	16.4	5.2	12.3	13.17	1.00	0.49	1.72
12	70	16430	16.4	5.4	12.0	13.17	1.00	0.48	1.75
13	70	16443	16.4	6.3	11.1	13.17	1.00	0.45	1.89
14	70	16456	16.4	6.4	11.0	13.17	1.00	0.44	1.91
15	70	16469	16.4	5.4	12.1	13.17	1.00	0.48	1.75
16	70	16482	16.4	5.1	12.4	13.17	1.00	0.50	1.70
17	70	16495	16.4	5.1	12.4	13.17	1.00	0.50	1.70
18	70	16508	16.4	5.4	12.0	13.17	1.00	0.48	1.75
19	70	16521	16.4	6.8	10.7	13.17	1.00	0.43	1.97
20	70	16534	16.4	6.0	11.5	13.17	1.00	0.46	1.83
21	70	16547	16.4	5.2	12.3	13.17	1.00	0.49	1.72
22	70	16560	16.4	4.9	12.5	13.17	1.00	0.50	1.69
23	70	16573	16.4	5.0	12.4	13.17	1.00	0.50	1.70
24	70	16586	16.4	5.5	11.9	13.17	1.00	0.48	1.77
25	70	16599	16.4	8.1	9.3	13.17	1.00	0.37	2.28
26	70	16612	16.4	5.7	11.8	13.17	1.00	0.47	1.79
27	70	16625	16.4	5.1	12.4	13.17	1.00	0.49	1.70
28	70	16638	16.4	5.0	12.5	13.17	1.00	0.50	1.69
29	70	16651	16.4	5.1	12.3	13.17	1.00	0.49	1.71
30	70	16664	16.4	5.8	11.6	13.17	1.00	0.46	1.81
31	70	16677	16.4	7.6	9.9	13.17	1.00	0.39	2.13
32	70	16690	16.4	5.8	11.7	13.17	1.00	0.47	1.80
33	70	16703	16.4	5.5	11.9	13.17	1.00	0.48	1.77
34	70	16716	16.4	6.1	11.4	13.17	1.00	0.45	1.85
35	70	16729	16.4	6.7	10.7	13.17	1.00	0.43	1.96
36	70	16742	16.4	5.2	12.2	13.17	1.00	0.49	1.72
37	70	16755	16.4	4.7	12.7	13.17	1.00	0.51	1.66
38	70	16768	16.4	4.7	12.7	13.17	1.00	0.51	1.66
39	70	16781	16.4	5.3	12.1	13.17	1.00	0.49	1.73
40	70	16794	16.4	5.9	11.6	13.17	1.00	0.46	1.82
41	70	16807	16.4	4.1	13.4	13.17	1.00	0.54	1.57


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 163+00 to 167+00
Critical Station = 167+55
Without Wells
With Wells
FSreq
Landside Toe
Bluff
Ground surface higher towards bluff than used to calculate FSreq
Head Above Ground Surface (ft)
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
Distance from Seepage Entrance Point (ft) (positive indicates landward)


CID-KS Feasibility Study Phase II
Hydraulic Grade Line Station 163+00 to 167+00
Landside Toe
Head Above Ground Surface (ft)
Station
Without Wells
With Wells
FSreq
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
16300
16400
16500
16600
16700
16800

Kansas Citys, Missouri and Kansas
Flood Risk Management Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Final Feasibility Report

Chapter A-5

GEOTECHNICAL ANALYSIS CID-MO

CHAPTER A-5
GEOTECHNICAL ANALYSIS – CID MO

A-5.1 INTRODUCTION

This chapter of the engineering appendix presents the results of the geotechnical evaluation performed for the Central Industrial District Levee Unit - Missouri (CID-MO). The evaluation started with a thorough review of existing project documentation, defining existing subsurface conditions along the entire unit based upon existing subsurface information, and estimation of soil parameters for the existing levees, the natural blanket and the aquifer materials. Additional subsurface investigations were performed to better define foundation materials for underseepage and foundation analyses. The estimated soil parameters are based on geotechnical laboratory testing data from adjacent projects since data was not readily available for the CID-MO Unit. Data was obtained from the North Kansas City Levee (across MO River), CID-KS Levee (upstream on KS River), East Bottoms Levee (downstream on MO River), and the Fairfax Jersey Creek Levee (upstream on MO River). All elevations used in the geotechnical portion of the feasibility study are NGVD 29 unless otherwise stated.

Geotechnical analysis of the unit consisted mainly of underseepage and foundation capacity calculations to support structural analysis of the shallow footing and pile founded floodwall for the existing level of flood protection, approximately N500+3.

Underseepage factors of safety were calculated along the entire CID-MO Unit. Since all areas met underseepage criteria, no reliability analysis was performed. Shallow footing and pile capacities were provided for structural analysis.

A-5.2 DESCRIPTION OF EXISTING LEVEE UNIT

A-5.2.1 Levee Description

The CID-MO Unit is located in Jackson County, Missouri and extends along the right bank of the Missouri River from RM 365.7, at the Kansas-Missouri state line and termination of the CID-KS Levee Unit (Station 83+01.29), downstream to RM 367.2, near the Grand St. viaduct (Station 0+00), where the floodwall terminates into high ground upstream of the East Bottoms Levee Unit. The CID-MO and CID-KS Units are directly connected and there is no hydraulic separation.

The unit's system consists of mostly floodwall with some levee segments, stoplog gaps, pumping plants, drainage structures, riprap and levee toe protection, and surfaced levee crown and ramps. The greater portion of the area is highly industrialized. These areas are occupied largely by railroads, wholesale houses, water treatment plants, and manufacturing plants. The total length of the unit is 8,301 feet or about 1.6 miles.

There are many bridges, structures, and utilities within the critical area of the line of protection. For the purposes of the geotechnical analysis, it was assumed that all bridge foundation elements, structures, and utilities within the levee embankment and critical area of the foundation blanket material meet all pertinent Corps of Engineers criteria. Henceforth, no analysis was completed regarding their integrity.

A-5.2.2 History

The Kansas Citys Flood Control Project, of which the Central Industrial Unit (Missouri Section) is a part, was authorized by Section 9 of the Flood Control Act approved 22 June 1936, Public Law 738, 74th Congress, 2d Session, as modified and extended by Section 10 of the Flood Control Act approved 22 December 1944, Public Law 634, 78th Congress, 2d Session.

Early flood protection works prior to Federal participation consisted of levees and retaining walls. Federal participation started with the Flood Control Act of 1936, and on 6 March 1946, a contract was awarded for the construction of levees, floodwalls, and appurtenances for the Central Industrial Unit (Missouri Section). Construction began on 21 March 1946 and was completed on 9 September 1947. Since that time, improvements have been made to the unit under other Corps of Engineers' contracts to construct the Broadway and Santa Fe Pumping Plants, restore the flood protection after the 1951 flood (CID-KS overtopping led to an exit overtopping and large scour hole near Station 80+00 by the stoplog gap), restore riverside slope protection after the 1951 food, construction of emergency gates and appurtenances, and minor scour repairs after the 1993 flood. The following discussion describes the existing unit in additional detail by major features in a downstream direction.

Station 83+01.29 to 80+54.12

This is a floodwall section that is a continuation of the CID-KS floodwall. This section was constructed on a large pervious fill. There is a buried collector system that extends from Station 78+00 and terminates at Station 5+00 CID-KS to collect underseepage through the fill.

Station 80+54.12 to 78+00

This section is a levee section with a stoplog gap at Station 80+19.

Station 78+00 to 0+00

This section is floodwall section with a landward toe drain. The floodwall is supported by driven concrete piles with a concrete riverside cut off wall from Station 78+00 to the Hannibal Bridge near Station 25+25. The floodwall is supported by a shallow footing bearing on bedrock between Stations 25+75 and 10+00. The floodwall is supported by a shallow footing on soil between Stations 10+00 and 0+00. There are gap structures at Stations 70+71, 68+90, 63+15, 14+80, 8+68, 5+24, and 1+53.

Based upon the record drawings, the existing levee sections are homogeneous embankments constructed of impervious fill.

A-5.2.3 General Geology of the Region (Missouri River)

The units are near the southern edge of the Dissected Till Plains section of the Central Lowlands Physiographic Province. The southern limit of glaciation in Missouri is generally considered to be just south of the Missouri River. During the Pleistocene, both the Nebraskan and Kansas glaciation crossed Platte County. The topography consists mainly of flat-lying alluvial sediments of the Missouri River floodplain, bounded by rolling hills comprising the valley walls. Maximum relief in the area is about 170-ft. The Missouri River alluvium generally ranges from approximately 110 to 130-ft in thickness, with the exception of buried stream channels that may extend into the Marmaton Group. All of the Missouri alluvium lies on shales and siltstones in the Pleasanton Group of the late Pennsylvanian System. The valley walls are composed of alternating layer of shales and limestone of the Kansas City Group. Drainage is by means of a maturely developed dendritic pattern except where it has been altered by human activity.

A-5.2.4 Subsurface Conditions

Assessments of the subsurface conditions for the project were derived from the Record Drawings, Design Memorandums and borings made at selected sites during Phase 1 and Phase 2 of the Feasibility Study. Typical subsurface conditions for the CID-MO Unit consist of a "two blanket" system. There is an upper blanket with typical thickness of 3 to 6-ft, underlain by an upper pervious layer with typical thickness of 8 to 20-ft, underlain by a lower blanket with typical thickness of 14 to 20-ft, underlain by the aquifer with typical thickness of 45 to 55-ft, underlain by bedrock. The upper and lower blankets appear to be connected riverward of the floodwall. It is hypothesized that lower blanket is the natural blanket, and the upper pervious layer and upper blanket were fill placed to raise the area during commercial development prior to construction of the floodwall. The floodwall does bear directly on bedrock between approximate Stations 10+00 and 25+75. Groundwater levels are dependent on seasonal changes and are generally equal to the Missouri River elevation.

A-5.3 UNDERSEEPAGE ANALYSIS

The Kansas City District method of estimating the hydraulic gradients due to underseepage is slightly different than the method described in the EM 1110-2-1913. It is based on the findings made at the Missouri River Division Conference held by the Corps of Engineers in 1962 in Omaha. The underseepage analysis was based on experience during the flood event in 1952 along the Missouri River. The main differences in the Kansas City District method are:

1. The Kansas City District Method uses permeability ratios (See Table A-5.1.) related to differing material types of the blanket material instead of using actual horizontal and vertical permeabilities.

2. The Kansas City District Method assumes an infinite landside blanket in the analysis.

3. The Kansas City District Method does not use a transformed thickness for the soil stratum considered as EM 1110-2-1913 allows, instead, a representative permeability ratio is applied to the overall blanket thickness.

For the underseepage analysis, the entire CID-MO Unit was divided into reaches of similar protection height, blanket thickness, blanket composition, aquifer thickness, and seepage entrance conditions. The factor of safety with respect to hydraulic gradient through the natural blanket was calculated for each of these reaches at the landside toe of the levee section or floodwall for a series of alternatives. Five alternative analysis methods were used to analyze the unique foundation conditions at the CID-MO area due to the upper and lower blankets and buried collector system. The floodwall toe drain was ignored in all analyses.

TABLE A-5.1
Permeability Ratios for Blanket Material Based on Material Type

Blanket Material	Assumed Permeability Ratio
SM	100
ML	200-400
ML - CL	400
CL	400-600
CH	800-1000

Analysis Alternative 1
Assumes the blanket that is resistant to underseepage forces is equal to the thickness of the upper and lower blanket thicknesses added together. This analysis, while the least conservative of all alternatives, is conservative in that it ignores the thickness of the upper pervious layer in between the two blankets. This analysis is thought to be the most realistic of alternatives 1-4.

Analysis Alternative 2
Assumes the blanket thickness that is resistant to underseepage forces is equal to only the upper blanket thickness. This analysis is the most conservative analysis and ignores the existence of the lower blanket completely.

Analysis Alternative 3
Assumes the blanket thickness that is resistant to underseepage forces is equal to only the upper blanket thickness and used the upper pervious layer as the aquifer. However, the analysis assumed that the upper and lower blankets are connected by the seepage cut off wall and/or the riverside tie in, and the hydraulic pressure head was reduced by 25%. This analysis is a conceptual check on the gradient through the upper blanket.

Analysis Alternative 4
Assumes the blanket thickness that is resistant to underseepage forces is equal to only the lower blanket thickness. The upper pervious layer is assumed to have hydrostatic conditions, and the aquifer is assumed to be surcharged by the river. This analysis is a conceptual check on the gradient through the lower blanket.

Analysis Alternative 5
The last alternative is an analysis that was used where the buried collector system exists where the levee and floodwall was constructed on the large pervious fill. The analysis assumes that the buried collector system maintains hydrostatic conditions in the upper pervious layer, and the lower blanket is the resistance to underseepage forces. This analysis is a check on the gradient through the lower blanket where the buried collector system exists.
Exhibit A-5.1, located at the end of the chapter, shows the calculated factor of safety with respect to hydraulic gradient for the entire CID-MO Levee Unit for all analysis alternative methods with water at the top of protection. The analysis shows all input parameters used to calculate the factor of safety. Supporting documentation consisting of riverside and centerline subsurface profiles and foundation cross sections are also included in Exhibit A-5.1. For all alternative analysis methods except for Alternative 2 indicate the levee will perform well for a top of wall loading. Calculated factors of safety are generally in excess of 1.6. This is in agreement with observations during the 1993 flood which reported no adverse seepage conditions with water 2 to 3-ft from the top of protection. Kansas City District Underseepage Criteria is discussed in the NWK Levee Underseepage Guidance in Exhibit A-5.2 at the end of the chapter.

A-5.4 SOIL STRENGTH PARAMETERS
The required parameters for soils in the CID-MO Unit were estimated mainly from the significant amount of geotechnical laboratory testing data performed for adjacent levee units. Little information regarding strength parameter development for the CID-MO Unit could be located for this study. Soil information from CID-KS, East Bottoms, Fairfax Jersey Creek, and North Kansas City levees were used. This information is located in Exhibit A-5.3 at the end of the chapter. A summary of the soil parameters is provided in Table A-5.2 below and discussed in the following paragraphs.

TABLE A-5.2
Geotechnical Design Parameters

Material	Unit Weight (pcf)		Drained Shear Strength		Undrained Shear Strength*	
	Moist	Saturated	ϕ' (degrees)	c' (psf)	ϕ (degrees)	c (psf)
Embankment	115	120	29	0	0	1000
Fill/Debris+	110	115	20	0	N/A	600
Foundation Blanket*	110	115	22	0	0	600
Foundation Sand	115	120	30	0	N/A	N/A

+Assumed parameters based on weakest perceived material likely to be present
*CH material not included

The blanket materials consist mostly of ML and CL materials, with some discontinuous layers of CH, SM, and unclassified fill material. The shear strength for the foundation sands was estimated from standard penetration test data performed in October 2001. The information used is considered adequate, if not conservative, for this study and is available upon request. These strength parameters were used in the structural analysis for floodwall stability.

A-5.5 FOUNDATION CAPACITY

A-5.5.1 Shallow Foundation Capacity

Shallow foundation bearing capacity was calculated using Vesic's bearing capacity factors for floodwall founded on soil and provided for structural analysis. Bearing capacity for floodwall founded on limestone was estimated using AASHTO HB-17, Table 4.11.4.1.4-1. Shallow foundation bearing capacity is shown in Exhibit A-5.4 at the end of this chapter. Generally bearing capacity does not control floodwall stability, as sliding stability is usually more critical. Additional discussion is located in the structural analysis chapter of this appendix.

A-5.5.2 Deep Foundation Capacity

Deep foundation capacity was calculated in general accordance with EM 1110-2-2906 *Design of Pile Foundations*. Capacity was calculated for drained and undrained conditions. However, drained conditions usually controlled the analysis. Skin friction resistance and tip resistance were calculated separately and added together to determine total pile axial compression capacity. Tensile capacity was taken as 70% of the compression skin friction resistance as recommended in Table 4-5 of EM 1110-2-2906. Earth pressure coefficients for skin resistance of 1.25 for clay and 2.0 for sand were also obtained from Table 4-5 for a high displacement driven pile. There was no reduction in soil-pile interaction friction angle. Additionally, the concept of a "critical depth" for drained analysis was not used even though it is specified in EM 1110-2-2906. This is because published work and other governmental agencies (FHWA) have determined that the concept of "critical depth" as stated in EM 1110-2-2906 is overly conservative. There is evidence that a limiting value of side and tip resistance is appropriate in some cases, but generally at pile depths greater than what are present at CID-MO.

All the bearing piles at CID-MO are square precast concrete driven piles. However, different lengths and sizes were used. Lengths varied between 21 and 34-ft and sizes varied between 16 and 18 inches. Additionally, the concrete cut off pile was considered for capacity. The concrete cut off pile is typically 16-ft long and 10 inches wide.

A summary of calculated pile capacities is shown below in Table A-5.3. The calculated capacities are in general agreement with capacity estimates from driving formulas during original floodwall construction. Refer to Exhibits A-5.5, Ultimate Design Pile Capacity Calculations and Summary; Exhibit A-5.6, Limestone Friction Angle Determination; Exhibit A-5.7, Pile Capacity Reliability; and Exhibit A-5.8, Floodwall Bearing Capacity at the end of this chapter for more information regarding this analysis. For further information on pile founded floodwall stability, see the structural analysis chapter of this appendix.

TABLE A-5.3
Ultimate Pile Capacity Summary

Station Start	Station Stop	Pile Size (inch)	Pile Shape	Pile Length (feet)	Axial Compressive Capacity (lb)	Axial Tensile Capacity (lb)	Axial Compressive Capacity (ton)	Driving Formula Estimate, Construction (ton)	Monolith Range
22+81.46	24+54.76	18	tapered	21	63,184	34,428	32	35	50-53
24+54.76	25+38.76	16	straight	25	94,544	47,066	47	40	54-55
25+38.76	26+22.76	16	straight	29	117,871	60,583	59	35+	56-57
26+22.76	27+90.76	16	straight	30	110,124	56,280	55	35+	58-61
22+81.46	27+90.76	10	cut off pile*	16	22,839	9,174	11	-	-
27+90.76	30+42.75	16	straight	34	165,730	93,113	83	50	62-67
30+42.75	32+52.76	16	straight	21	87,472	44,990	44	50+	68-72
27+90.76	30+42.75	10	cut off pile*	16	34,658	7,421	17	-	-
32+52.76	48+06.76	16	straight	21	118,051	67,168	59	35-50	73-109
32+52.76	48+06.76	10	cut off pile*	16	46,675	15,817	23	-	-
48+06.76	60+24.76	18	tapered	21	76,858	45,490	38	35-50	110-138
48+06.76	60+24.76	10	cut off pile*	16	28,969	14,351	14	-	-
60+24.76	73+20.14	18	straight	34	137,963	73,264	69	40-50+	139-167
73+20.14	78+12.22	16	straight	21	67,405	34,166	34	30	168-179
60+24.76	78+12.22	10	cut off pile*	16	20,730	9,401	10	-	-

*cut off pile capacities are in lb/ft

A-5.6 HYDRAULIC GRADE LINES FOR PUMP PLANTS

In addition to the overall underseepage analysis performed for the CID Missouri Unit, the underseepage conditions were also evaluated specifically for all pump plants using the underseepage criteria discussed previously. The underseepage was evaluated for existing conditions, the N500+0, N500+3 and N500+5 river levels. For feasibility level detail, the cross section of the protection was not changed to reflect changes in levee geometry required for the raises. However, the results of this analysis should reasonably reflect the underseepage conditions for river levels higher than existing conditions, and should be conservative.

The purpose of the analysis was to provide the structural engineers with an excess head value at the base of the natural blanket for the different river levels at each pump plant location. To accomplish this, hydraulic grade lines were computed at each pump plant location for each river level. The pump plant locations are identified in Table A-5.4 on the next page. The results are provided in Exhibit A-14.2 at the end of Chapter A-14 of this Engineering Appendix.

TABLE A-5.4
Pump Plants Analyzed for Underseepage

Pump Plant	Approximate Station
Broadway	24+76.9
Santa Fe	52+86.7
Kemper Arena	106+49 (KS Stationing)

For the Broadway and Santa Fe pump stations, the analyses were difficult due to the lack of information. The intent of the analyses was to err on the side of conservatism due to the considerable unknowns. The lack of information is likely due to the original need for only shallow information for pile design. The current analysis indicates significant potential for underseepage problems. For PED the following unknowns need to be addressed to adequately address underseepage at these locations:

1. The top of bedrock is unknown. For the Broadway plant it appears the bedrock is shallow and quickly dropping off, and for Santa Fe none of the existing borings were drilled to bedrock.

2. Thickness of the aquifer is not defined. None of the existing borings were drilled to a depth where any existing natural aquifer could be identified.

3. Definition of the blanket. From the available information there appears to have been a significant amount of highly heterogeneous fill placed over the site. Materials include debris, cinders, sand, loam, concrete, and other soil materials.

To address these unknowns it is strongly recommended that a comprehensive drilling effort be undertaken.

At the Kemper Arena site there is also a significant amount of debris fill (mostly described as cinders), however it is better defined by previous subsurface investigations. For this analysis the top of the fill was considered the landside ground surface, however, the thickness of the fill was subtracted from the thickness of the blanket due to the unknown fill materials permeability characteristics.

A-5.7 REFERENCES

1. Operations and Maintenance Manual, Kansas City Flood Control Project, Missouri and Kansas River, Central Industrial Unit - Missouri Section, Volume I, 1981.

2. Operations and Maintenance Manual, Record Drawings, Kansas City Flood Control Project, Missouri and Kansas River, Central Industrial Unit Missouri Section, Volume I, Appendix I, Dated 1944 - 1957.

3. Corps of Engineers Engineering Manuals, Technical Letters, etc. as referenced within.

A-5.8 SUPPLEMENTAL EXHIBITS

EXHIBIT A-5.1

CID MO – Underseepage Calculations

CID-MO Levee Kansas City Phase 2 Feasibility
Saturday, August 08, 2009 Revised:
Levee Foundation Information - EXISTING CONDITIONS, Water to Top of Levee

Blanket Unit Weight = 115.0 pcf (saturated)
Rubble Fill Unit Weight = 115.0 pcf (saturated)

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Blanket Elevation (msl)	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio River Side k_f/k_{bl}	Permeability Ratio Land Side k_f/k_{bl}	Impervious Blanket Thickness (ft) River Side z_{bl}	Impervious Blanket Thickness (ft) Levee z_{bl}	Impervious Blanket Thickness (ft) Land Side z_{bl}	Pervious Blanket Thickness (ft) d
0+00 to 10+00	758.0	750	752	711	725	749	710	6.0	300	300	39	31.5	24	15
10+00 to 22+50	758.5	750	750	715	floodwall sitting on bedrock	750	735	8.5	300	300	35	#VALUE!	#VALUE!	#VALUE!
22+50 to 27+50	759.0	750	752	716	710	742	700	7.0	300	300	34	33	32	10
27+50 to 32+50 - Assumption 1	759.3	750	752	716	710	736	690	7.3	300	300	34	29.5	25	20
27+50 to 32+50 - Assumption 2	759.3	750	752	716	740	748	690	7.3	300	300	5	6.5	8	50
27+50 to 32+50 - Assumption 3	759.3	750	752	716	740	748	690	5.4	300	300	5	6.5	8	14
27+50 to 32+50 - Assumption 4	759.3	750	752	716	710	722	690	7.3	300	300	34	23	12	20
32+50 to 37+50 - Assumption 1	759.4	750	752	720	710	730	665	7.4	300	300	30	25	20	45
32+50 to 37+50 - Assumption 2	759.4	750	752	720	745	752	665	7.4	300	300	30	18.5	7	80
32+50 to 37+50 - Assumption 3	759.4	750	752	720	745	752	665	5.5	300	300	5	6	7	22
32+50 to 37+50 - Assumption 4	759.4	750	752	720	710	724	665	7.4	300	300	30	22	14	45
37+50 to 41+00 - Assumption 1	759.4	750	752	721	709	733	665	7.4	300	300	29	26.5	24	44
37+50 to 41+00 - Assumption 2	759.4	750	752	721	746	752	665	7.4	300	300	29	17.5	6	81
37+50 to 41+00 - Assumption 3	759.4	750	752	721	746	752	665	5.6	300	300	3	4.5	6	20
37+50 to 41+00 - Assumption 4	759.4	750	752	721	709	727	665	7.4	300	300	29	23.5	18	44

Station	Seepage Length (ft) River Side L_1	Seepage Length (ft) Levee L_2	Seepage Length (ft) Land Side L_3	Factor c_1	Factor c_3	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Factor of Safety for Piping (i_c/i_o)	Remarks
0+00 to 10+00	100	15	1000	0.002387	0.003043	98	329	4.46	0.19	0.84	4.53	
10+00 to 22+50	100	15	1000	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	0.84	#VALUE!	floodwall is bearing directly on bedrock and there is a toe drain at the landward edge of the floodwall. A line of creep analysis, or similar, will have to be performed if the uplift along the floodwall is needed.
22+50 to 27+50	100	15	1000	0.003131	0.003227	97	310	5.14	0.16	0.84	5.24	
27+50 to 32+50 - Assumption 1	100	15	1000	0.002214	0.002582	98	387	5.61	0.22	0.84	3.76	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
27+50 to 32+50 - Assumption 2	50	15	1000	0.003651	0.002887	49	346	6.11	0.76	0.84	1.10	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
27+50 to 32+50 - Assumption 3	50	15	1000	0.006901	0.005456	437	183	1.57	0.20	0.84	4.30	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 25% in the upper sand. This calculation checks the gradient through the upper blanket.
27+50 to 32+50 - Assumption 4	100	15	1000	0.002214	0.003727	98	268	5.10	0.42	0.84	1.98	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.
32+50 to 37+50 - Assumption 1	100	15	1000	0.001571	0.001925	99	520	6.07	0.30	0.84	2.78	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
32+50 to 37+50 - Assumption 2	50	15	1000	0.001179	0.002440	50	410	6.39	0.91	0.84	0.92	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
32+50 to 37+50 - Assumption 3	50	15	1000	0.005606	0.004652	677	215	1.32	0.19	0.84	4.48	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 25% in the upper sand. This calculation checks the gradient through the upper blanket.
32+50 to 37+50 - Assumption 4	100	15	1000	0.001571	0.002300	99	436	5.86	0.42	0.84	2.01	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.
37+50 to 41+00 - Assumption 1	100	15	1000	0.001616	0.001777	99	563	6.19	0.26	0.84	3.27	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
37+50 to 41+00 - Assumption 2	50	15	1000	0.001191	0.002619	50	382	6.36	1.06	0.84	0.80	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
37+50 to 41+00 - Assumption 3	50	15	1000	0.007454	0.005270	377	190	1.82	0.30	0.84	2.78	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 25% in the upper sand. This calculation checks the gradient through the upper blanket.
37+50 to 41+00 - Assumption 4	100	15	1000	0.001616	0.002052	99	487	6.03	0.33	0.84	2.52	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Blanket Elevation (msl)	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio River Side k_f/k_{br}	Permeability Ratio Land Side k_f/k_{bl}	Impervious Blanket Thickness (ft) River Side z_{br}	Impervious Blanket Thickness (ft) Levee z_{bb}	Impervious Blanket Thickness (ft) Land Side z_{bl}	Pervious Blanket Thickness (ft) d	Seepage Length (ft) River Side L_1	Seepage Length (ft) Levee L_2	Seepage Length (ft) Land Side L_3	Factor c_r	Factor c_l	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Factor of Safety for Piping (i_c/i_o)	Remarks
41+00 to 46+00 - Assumption 1	759.5	750	752	722	712	734	665	7.5	300	300	28	26	22	47	100	15	1000	0.001592	0.001795	99	557	8.22	0.28	0.84	2.98	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
41+00 to 46+00 - Assumption 2	759.5	750	752	722	746	752	665	7.5	300	300	28	17	6	81	50	15	1000	0.001212	0.002619	50	382	8.41	1.07	0.84	0.79	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
41+00 to 46+00 - Assumption 3	759.5	750	752	722	746	752	665	5.6	300	300	5	5.5	6	18	50	15	1000	0.006455	0.005893	497	170	1.40	0.23	0.84	3.61	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 25% in the upper sand. This calculation checks the gradient through the upper blanket.
41+00 to 46+00 - Assumption 4	759.5	750	752	722	709	725	665	7.5	300	300	28	22	16	44	100	15	1000	0.001645	0.002176	99	460	8.01	0.38	0.84	2.24	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.
46+00 to 50+00 - Assumption 1	759.6	750	752	718	713	733	665	7.6	300	300	32	26	20	48	100	15	1000	0.001473	0.001863	99	537	8.27	0.31	0.84	2.69	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
46+00 to 50+00 - Assumption 2	759.6	750	752	718	746	752	665	7.6	300	300	32	19	6	81	50	15	1000	0.001134	0.002619	50	382	6.50	1.08	0.84	0.78	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
46+00 to 50+00 - Assumption 3	759.6	750	752	718	746	752	665	5.7	300	300	5	5.5	6	18	50	15	1000	0.006086	0.005556	567	180	1.37	0.23	0.84	3.70	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 25% in the upper sand This calculation checks the gradient through the upper blanket.
46+00 to 50+00 - Assumption 4	759.6	750	752	718	713	729	665	7.6	300	300	32	24	16	48	100	15	1000	0.001473	0.002083	99	480	8.14	0.38	0.84	2.20	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.
50+00 to 52+50 - Assumption 1	759.7	750	750	715	712	736	665	9.6	300	300	35	29.5	24	47	100	15	1000	0.001423	0.001719	99	592	8.06	0.34	0.84	2.51	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
50+00 to 52+50 - Assumption 2	759.7	750	750	715	745	750	665	9.6	300	300	35	20	5	80	50	15	1000	0.001091	0.002887	50	349	8.13	1.63	0.84	0.52	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
50+00 to 52+50 - Assumption 3	759.7	750	750	715	745	750	665	4.8	300	300	5	5	5	14	50	15	1000	0.006901	0.006901	437	145	1.17	0.23	0.84	3.60	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 50% in the upper sand since it is a full cut off. This calculation checks the gradient through the upper blanket.
50+00 to 52+50 - Assumption 4	759.7	750	750	715	712	731	665	9.6	300	300	35	27	19	47	100	15	1000	0.001423	0.001932	99	518	7.90	0.42	0.84	2.03	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.
52+50 to 55+00 - Assumption 1	759.7	750	750	715	711.5	738.5	665	9.7	300	300	35	31	27	46.5	100	15	1000	0.001431	0.001829	99	614	8.18	0.30	0.84	2.78	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
52+50 to 55+00 - Assumption 2	759.7	750	750	715	744	750	665	9.7	300	300	35	20.5	6	79	50	15	1000	0.001098	0.002652	50	377	9.27	1.38	0.84	0.61	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
52+50 to 55+00 - Assumption 3	759.7	750	750	715	744	750	665	4.9	300	300	5	5.5	6	10	50	15	1000	0.008165	0.007454	316	134	1.40	0.23	0.84	3.62	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 50% in the upper sand since it is a full cut off. This calculation checks the gradient through the upper blanket.
52+50 to 55+00 - Assumption 4	759.7	750	750	715	711.5	732.5	665	9.7	300	300	35	28	21	46.5	100	15	1000	0.001431	0.001848	99	541	8.01	0.38	0.84	2.21	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Blanket Elevation (msl)	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio		Impervious Blanket Thickness (ft)			Pervious Blanket Thickness (ft) d	Seepage Length (ft)			Factor		Effective Seepage Length (ft)		Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Factor of Safety for Piping (i_c/i_o)	Remarks
									River Side k_f/k_{br}	Land Side k_f/k_{bl}	River Side z_{br}	Levee z_{bs}	Land Side z_{bl}		River Side L_1	Levee L_2	Land Side L_3	c_r	c_l	River Side x_1	Land Side x_3					
55+00 to 57+50 - Assumption 1	759.8	750	750	714.5	711	737.5	665	9.8	300	300	35.5	31	26.5	48	100	15	1000	0.001429	0.001654	99	605	8.24	0.31	0.84	2.71	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
55+00 to 57+50 - Assumption 2	759.8	750	750	714.5	745	750	665	9.8	300	300	35.5	20.25	5	80	50	15	1000	0.001083	0.002887	50	346	8.25	1.65	0.84	0.51	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
55+00 to 57+50 - Assumption 3	759.8	750	750	714.5	745	750	665	4.9	300	300	5	5	5	12	50	15	1000	0.007454	0.007454	377	134	1.28	0.25	0.84	3.37	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 50% in the upper sand since it is a full cut off. This calculation checks the gradient through the upper blanket.
55+00 to 57+50 - Assumption 4	759.8	750	750	714.5	711	732.5	665	9.8	300	300	35.5	28.5	21.5	48	100	15	1000	0.001429	0.001836	99	545	8.10	0.38	0.84	2.24	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand
57+50 to 62+50 - Assumption 1	760.0	750	748	715	714	738	665	12.0	300	300	35	28.5	22	49	100	15	1000	0.001394	0.001758	99	580	9.99	0.45	0.84	1.86	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
57+50 to 62+50 - Assumption 2	760.0	750	748	715	745	748	665	12.0	300	300	35	19	3	80	50	15	1000	0.001081	0.003727	50	298	9.68	3.22	0.84	0.26	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
57+50 to 62+50 - Assumption 3	760.0	750	748	715	745	748	665	8.0	300	300	5	4	3	12	50	15	1000	0.007454	0.009623	377	104	1.26	0.42	0.84	2.01	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 50% in the upper sand since it is a full cut off. This calculation checks the gradient through the upper blanket.
57+50 to 62+50 - Assumption 4	760.0	750	748	715	714	733	665	12.0	300	300	35	27	19	49	100	15	1000	0.001394	0.001892	99	528	9.87	0.52	0.84	1.62	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.
62+50 to 65+00 - Assumption 1	760.0	750	748	717	712	738	665	12.0	300	300	33	29.5	26	47	100	15	1000	0.001466	0.001652	99	605	10.09	0.39	0.84	2.17	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
62+50 to 65+00 - Assumption 2	760.0	750	748	717	744	748	665	12.0	300	300	33	18.5	4	79	50	15	1000	0.001131	0.003248	50	308	9.91	2.48	0.84	0.34	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
62+50 to 65+00 - Assumption 3	760.0	750	748	717	744	748	665	8.0	300	300	5	4.5	4	8	50	15	1000	0.009129	0.010206	256	98	1.59	0.40	0.84	2.12	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 50% in the upper sand since it is a full cut off. This calculation checks the gradient through the upper blanket.
62+50 to 65+00 - Assumption 4	760.0	750	748	717	712	734	665	12.0	300	300	33	27.5	22	47	100	15	1000	0.001466	0.001795	99	557	9.96	0.45	0.84	1.86	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.
65+00 to 67+50 - Assumption 1	760.0	750	750	715	709	742	665	10.0	300	300	35	34	33	44	140	15	1000	0.001471	0.001515	138	660	8.12	0.25	0.84	3.43	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
65+00 to 67+50 - Assumption 2	760.0	750	750	715	744	750	665	10.0	300	300	35	20.5	6	79	100	15	1000	0.001098	0.002652	100	377	7.67	1.28	0.84	0.66	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
65+00 to 67+50 - Assumption 3	760.0	750	750	715	744	750	665	5.0	300	300	6	5.5	6	7	100	15	1000	0.009759	0.008909	136	112	2.13	0.35	0.84	2.38	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 50% in the upper sand since it is a full cut off. This calculation checks the gradient through the upper blanket.
65+00 to 67+50 - Assumption 4	760.0	750	750	715	709	736	665	10.0	300	300	35	31	27	44	140	15	1000	0.001471	0.001675	138	597	7.98	0.29	0.84	2.86	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Blanket Elevation (msl)	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio River Side k_f/k_{br}	Permeability Ratio Land Side k_f/k_{bl}	Impervious Blanket Thickness (ft) River Side z_{br}	Impervious Blanket Thickness (ft) Levee z_{bo}	Impervious Blanket Thickness (ft) Land Side z_{bl}	Pervious Blanket Thickness (ft) d	Seepage Length (ft) River Side L_1	Seepage Length (ft) Levee L_2	Seepage Length (ft) Land Side L_3	Factor c_1	Factor c_3	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_2	Computed Hydraulic Gradient i_c	Critical Hydraulic Gradient i_c	Factor of Safety for Piping (i_c/i_c)	Remarks
67+50 to 71+00 - Assumption 1	760.0	750	750	719	706	743	665	10.0	300	300	31	34	37	41	140	15	1000	0.001619	0.001482	138	675	8.15	0.22	0.84	3.82	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
67+50 to 71+00 - Assumption 2	760.0	750	750	719	744	750	665	10.0	300	300	31	18.5	6	79	100	15	1000	0.001167	0.002652	100	377	7.67	1.28	0.84	0.66	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
67+50 to 71+00 - Assumption 3	760.0	750	750	719	744	750	665	5.0	300	300	5	5.5	6	10	100	15	1000	0.008165	0.007454	182	134	2.03	0.34	0.84	2.50	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 50% in the upper sand since it is a full cut off. This calculation checks the gradient through the upper blanket.
67+50 to 71+00 - Assumption 4	760.0	750	750	719	706	737	665	10.0	300	300	31	31	31	41	140	15	1000	0.001619	0.001619	138	617	8.02	0.26	0.84	3.26	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.
71+00 to 75+00 - Assumption 1	760.0	750	750	738	704	729	665	10.0	300	300	12	18.5	25	39	140	15	1000	0.002669	0.001849	134	541	7.84	0.31	0.84	2.69	Lower and upper blanket thickness added together to compute Zbl and FSi. This assumes that the riverside has an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This is the LEAST conservative approach
71+00 to 75+00 - Assumption 2	760.0	750	750	738	740	750	665	10.0	300	300	12	11	10	75	100	15	1000	0.001926	0.002108	99	474	8.07	0.81	0.84	1.05	Only the upper blanket thickness was used to compute Zbl and FSi. This assumes that the riverside does not have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall does not provide a cut off between the river and the middle sand. This is the MOST conservative approach
71+00 to 75+00 - Assumption 3	760.0	750	750	738	740	750	665	7.5	300	300	5	7.5	10	20	100	15	1000	0.005774	0.004082	333	245	3.10	0.31	0.84	2.72	Only the upper blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand that is used to calculate x1. the concrete sheetpile cutoff wall reduces the head by 50% in the upper sand since it is a full cut off. This calculation checks the gradient through the upper blanket.
71+00 to 75+00 - Assumption 4	760.0	750	750	719	704	718	665	10.0	300	300	31	22.5	14	39	140	15	1000	0.001660	0.002471	138	405	7.26	0.52	0.84	1.62	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.
75+00 to 78+00 - Assumption 5	760.0	750	750	749	704	720	665	10.0	300	300	1	8.5	16	39	0	75	1000	0.009245	0.002311	0	433	8.52	0.53	0.84	1.58	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does have an impervious layer over the middle sand and/or the concrete sheetpile cutoff wall provides a satisfactory cut off between the river and the middle sand to make pressures in the upper sand hydrostatic. This calculation checks the gradient through the lower blanket. Assumes hydrostatic conditions in the upper sand.
78+00 to 80+00 - Assumption 5	760.0	750	750	749	701	726	665	10.0	300	300	1	13	25	36	0	75	1000	0.009623	0.001925	0	520	8.74	0.35	0.84	2.41	Only the lower blanket thickness was used to compute Zbl and FSi. A large impervious section cuts off the middle sand. Also, the buried collector system maintains the pressure in the sand landward at hydrostatic. This is the likely scenario for this section.
80+00 to 82+50 - Assumption 5	760.0	750	750	749	703	726	665	10.0	300	300	1	12	23	38	0	75	1000	0.009366	0.001953	0	512	8.72	0.38	0.84	2.22	Only the lower blanket thickness was used to compute Zbl and FSi. A large impervious section cuts off the middle sand. Also, the buried collector system maintains the pressure in the sand landward at hydrostatic. This is the likely scenario for this section.
82+50 to 85+00 - Assumption 5	761.0	750	750	749	711	728	665	11.0	300	300	1	9	17	48	0	75	1000	0.008513	0.002085	0	484	9.53	0.56	0.84	1.50	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does not have an impervious layer over the middle sand but the buried collector system maintains the pressure in the sand landward at hydrostatic. This is the likely scenario for this section.
85+00 to 87+50 - Assumption 5	761.0	750	753	749	717	730	665	8.0	300	300	1	7	13	52	0	75	1000	0.008006	0.002221	0	450	6.86	0.53	0.84	1.60	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does not have an impervious layer over the middle sand but the buried collector system maintains the pressure in the sand landward at hydrostatic. This is the likely scenario for this section.
87+50 to 89+00 - Assumption 5	761.0	750	754	749	725	733	665	7.0	300	300	1	4.5	8	60	0	75	1000	0.007454	0.002635	0	379	5.84	0.73	0.84	1.15	Only the lower blanket thickness was used to compute Zbl and FSi. It was assumed that the riverside does not have an impervious layer over the middle sand but the buried collector system maintains the pressure in the sand landward at hydrostatic. This is the likely scenario for this section.

Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee width L_2

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground surface, feet
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$ $i_o = h_o/z_{bl}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$ $i_c = \gamma_b/\gamma_w$

$x_1 = \tanh(c_r L_1)/c_r$ $h_o = H((x_3/(x_1+L_2+x_3))$

$x_3 = 1/c_l$ $h_x = h_o e^{-cx}$

for calculations with bluff as seepage block:

$x_3 = 1/\ (c_l \tanh\ (c_l\ L3)$

for calculations with a seepage block at entrance

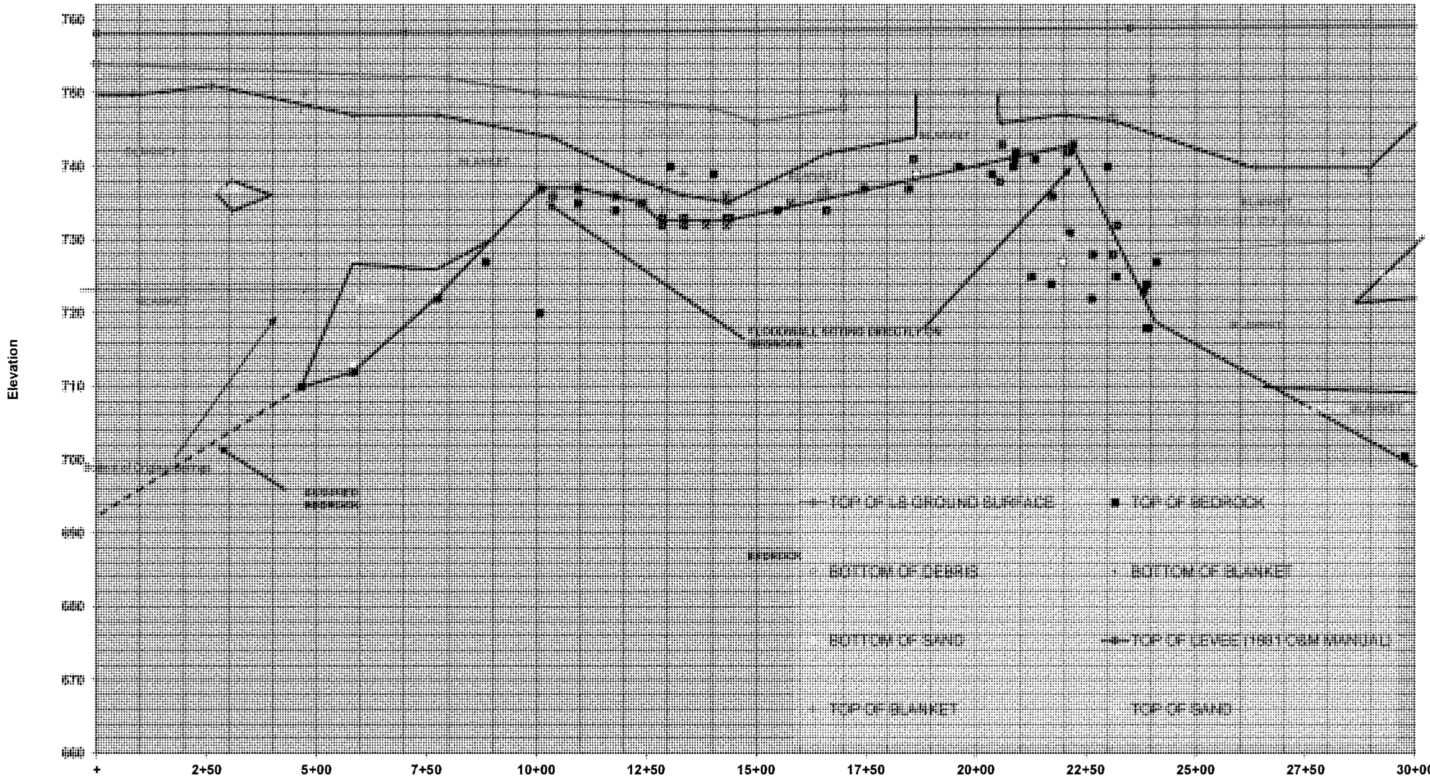
$c_r = (k_{br}/k_f z_{br} d)^{1/2}$ zbr = block thickness
d = upper aquifer thickness

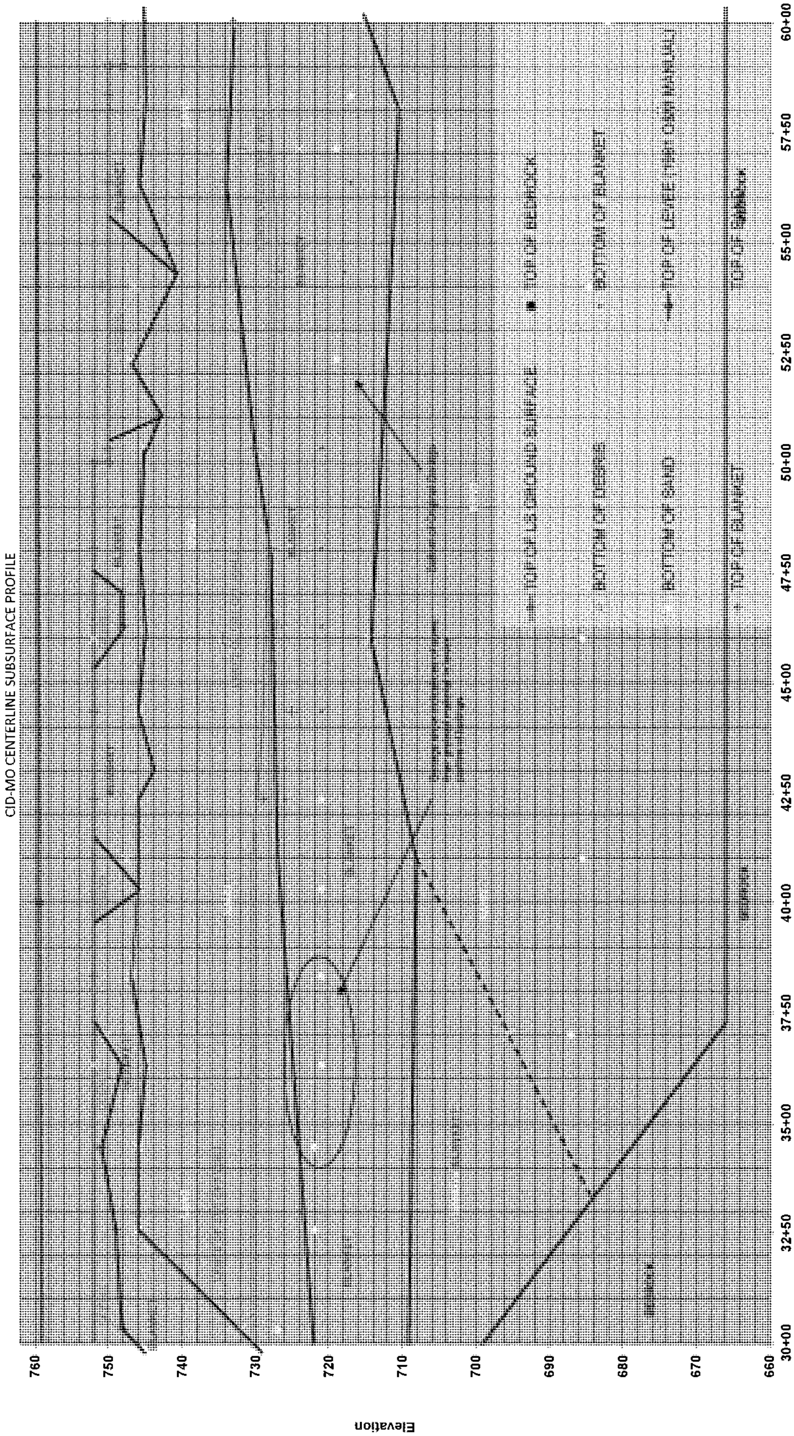
$x_1 = 1/(c_r \tanh\ (cL_1))$

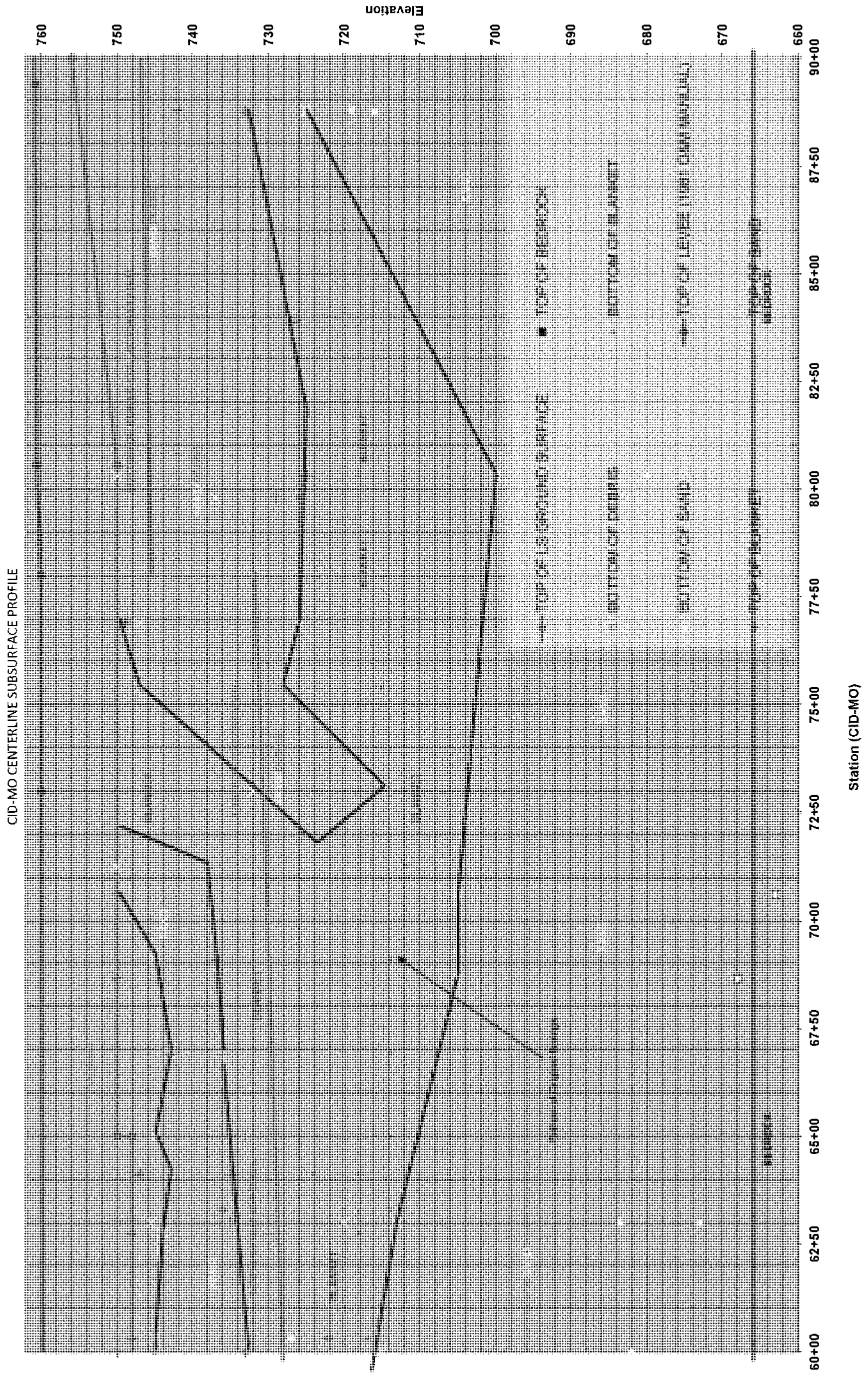
L1= distance to seepage block

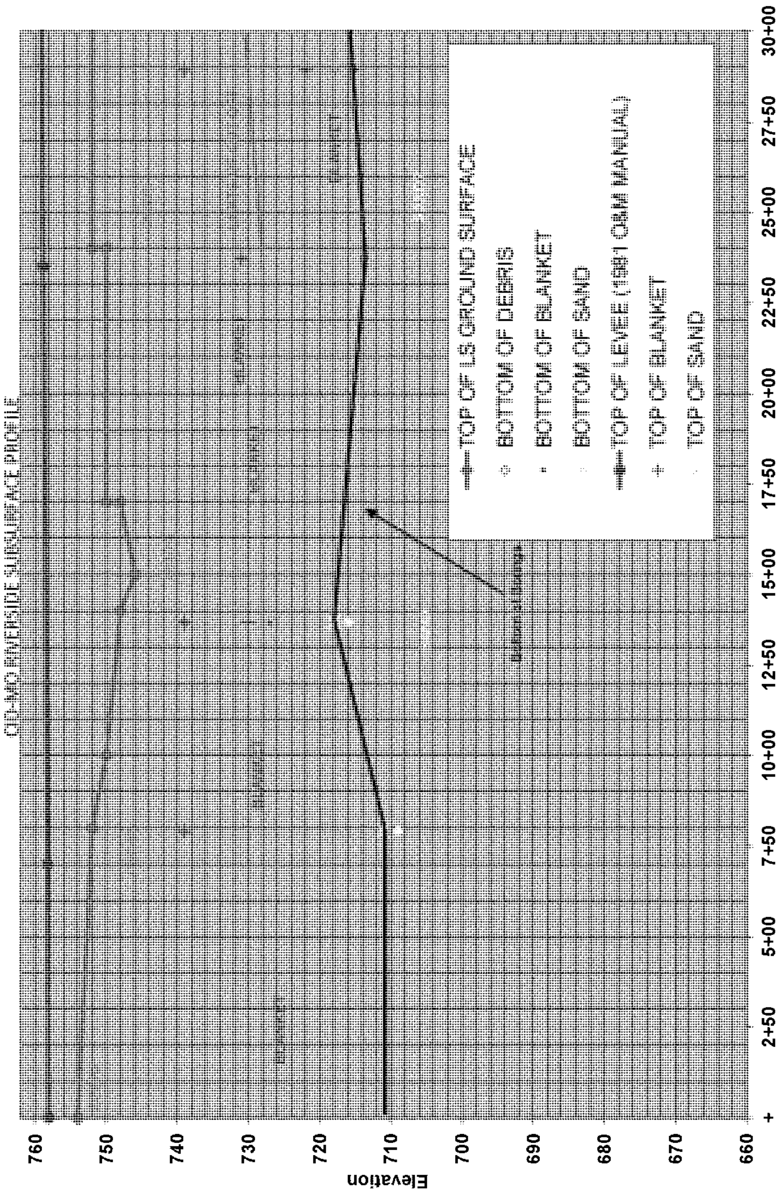
Designer	Glen M Bellew, PE(MO)	
Peer Review	Charlie Detrick	5-Jul-09
Evaluation	Glen M Bellew, PE(MO)	
Backcheck	Charlie Detrick	27-Jul-09

CID-MO CENTERLINE SUBSURFACE PROFILE

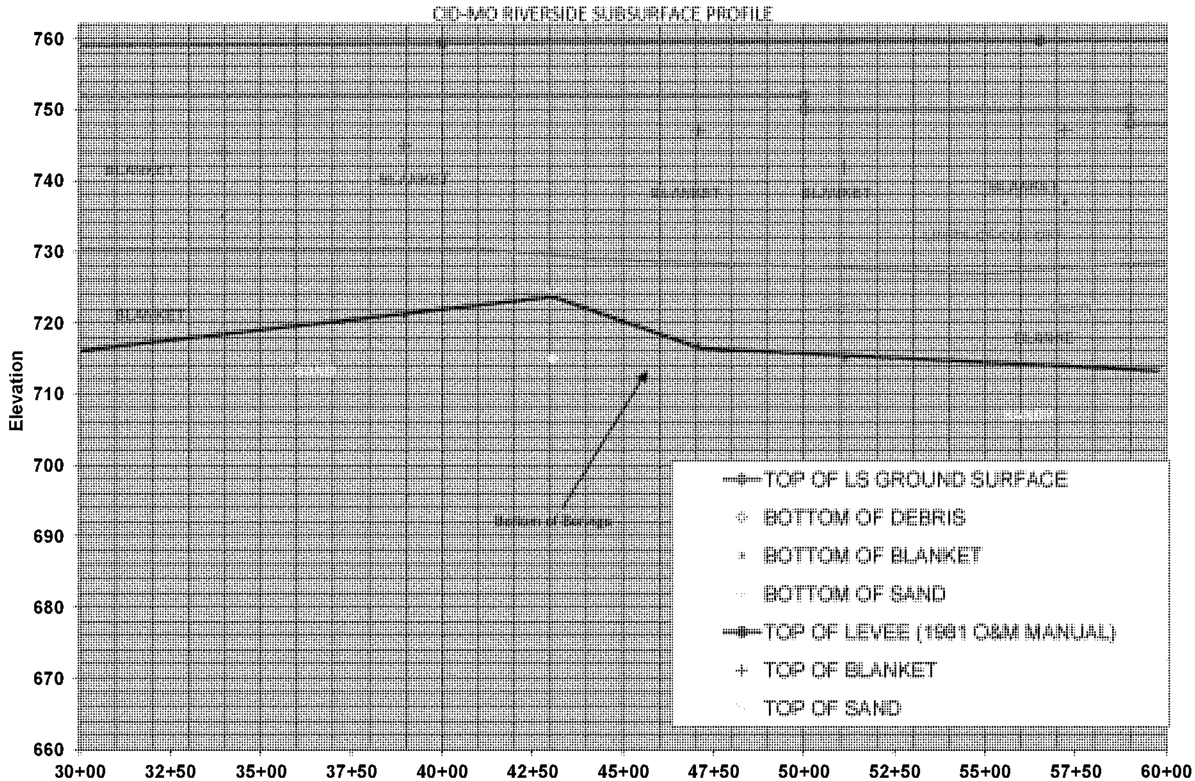


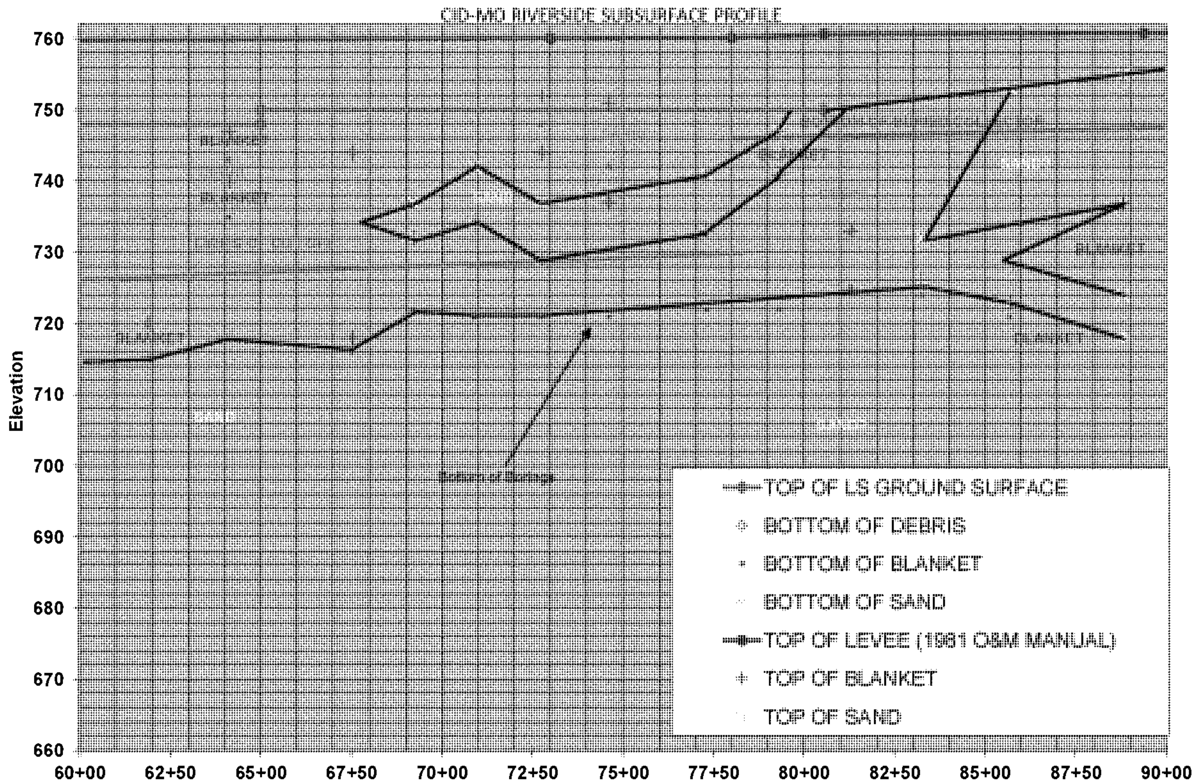

CID-MO CENTERLINE SUBSURFACE PROFILE
Elevation
760
750
740
730
720
710
700
690
680
670
660
30+00
32+50
35+00
37+50
40+00
42+50
45+00
47+50
50+00
52+50
55+00
57+50
60+00
TOP OF U.S. GROUND SURFACE
TOP OF BEDROCK
BOTTOM OF DEBRIS
BOTTOM OF BLANKET
BOTTOM OF SAND
TOP OF BLANKET


CID-MO CENTERLINE SUBSURFACE PROFILE
Elevation
760
750
740
730
720
710
700
690
680
670
660
Station (CID-MO)
60+00
62+50
65+00
67+50
70+00
72+50
75+00
77+50
80+00
82+50
85+00
87+50
90+00

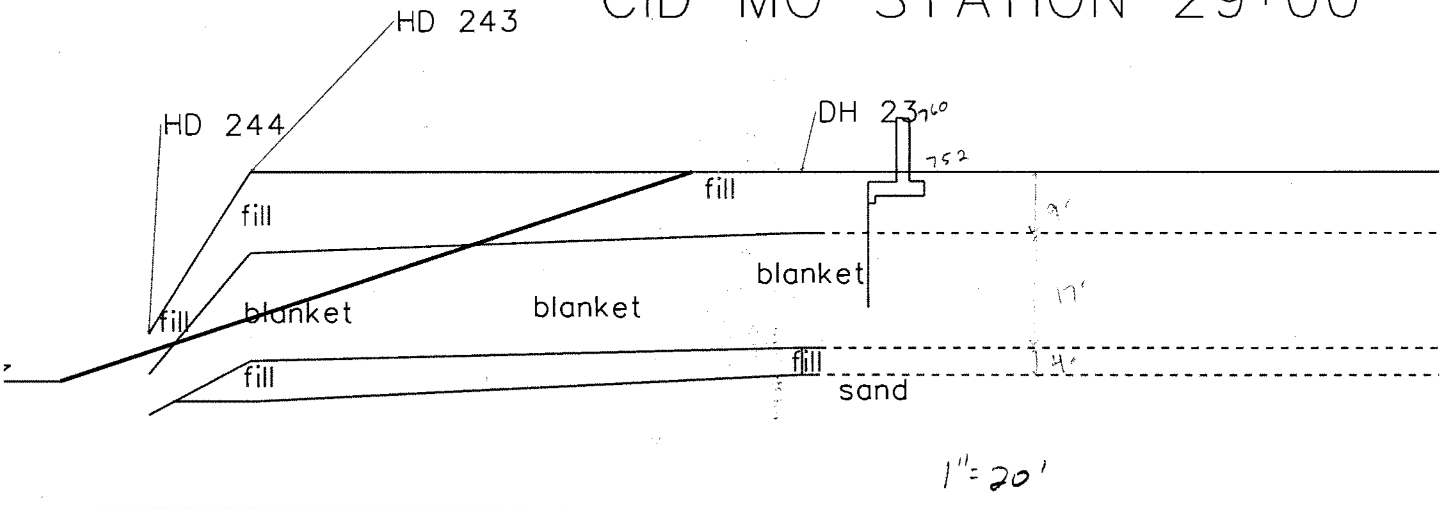


Elevation
760
750
740
730
720
710
700
690
680
670
660
+
2+50
5+00
7+50
10+00
12+50
15+00
17+50
20+00
22+50
25+00
27+50
30+00
TOP OF LS GROUND SURFACE
BOTTOM OF DEBRIS
BOTTOM OF BLANKET
BOTTOM OF SAND
TOP OF LEVEE (1981 O&M MANUAL)
TOP OF BLANKET
TOP OF SAND

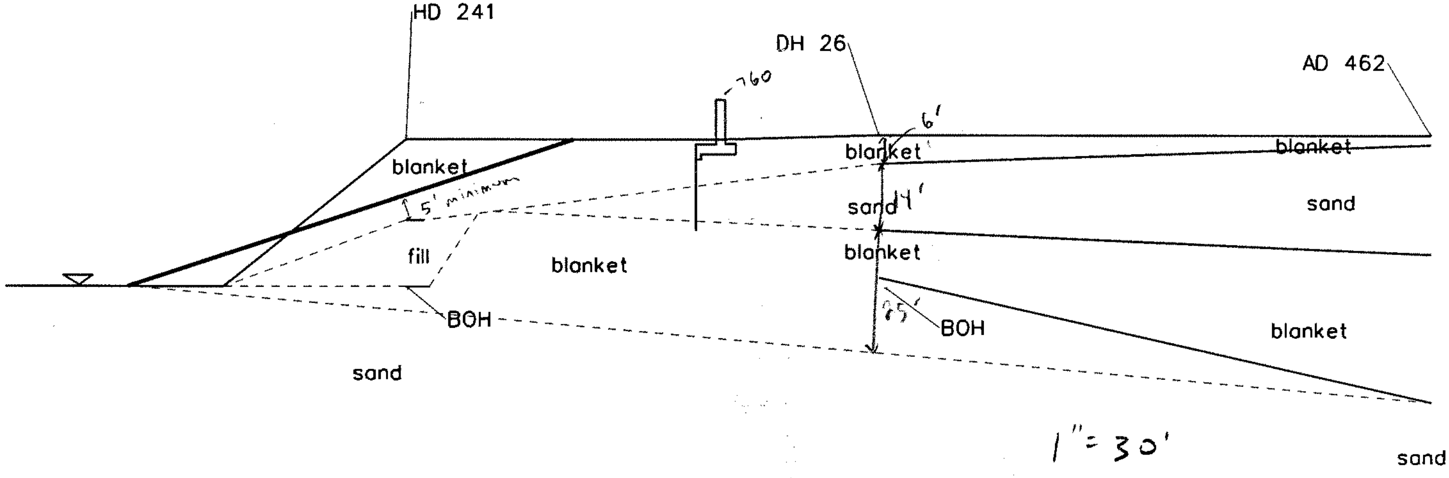

RIVERSIDE SUBSURFACE PROFILE
Elevation
760
750
740
730
720
710
700
690
680
670
660
30+00
32+50
35+00
37+50
40+00
42+50
45+00
47+50
50+00
52+50
55+00
57+50
60+00
TOP OF LS GROUND SURFACE
BOTTOM OF DEBRIS
BOTTOM OF BLANKET
BOTTOM OF SAND
TOP OF LEVEE (1961 O&M MANUAL)
TOP OF BLANKET
TOP OF SAND


CID-MO RIVERSIDE SUBSURFACE PROFILE
Elevation
760
750
740
730
720
710
700
690
680
670
660
60+00
62+50
65+00
67+50
70+00
72+50
75+00
77+50
80+00
82+50
85+00
87+50
90+00
Bottom of Borings
TOP OF LS GROUND SURFACE
BOTTOM OF DEBRIS
BOTTOM OF BLANKET
BOTTOM OF SAND
TOP OF LEVEE (1981 O&M MANUAL)
TOP OF BLANKET
TOP OF SAND

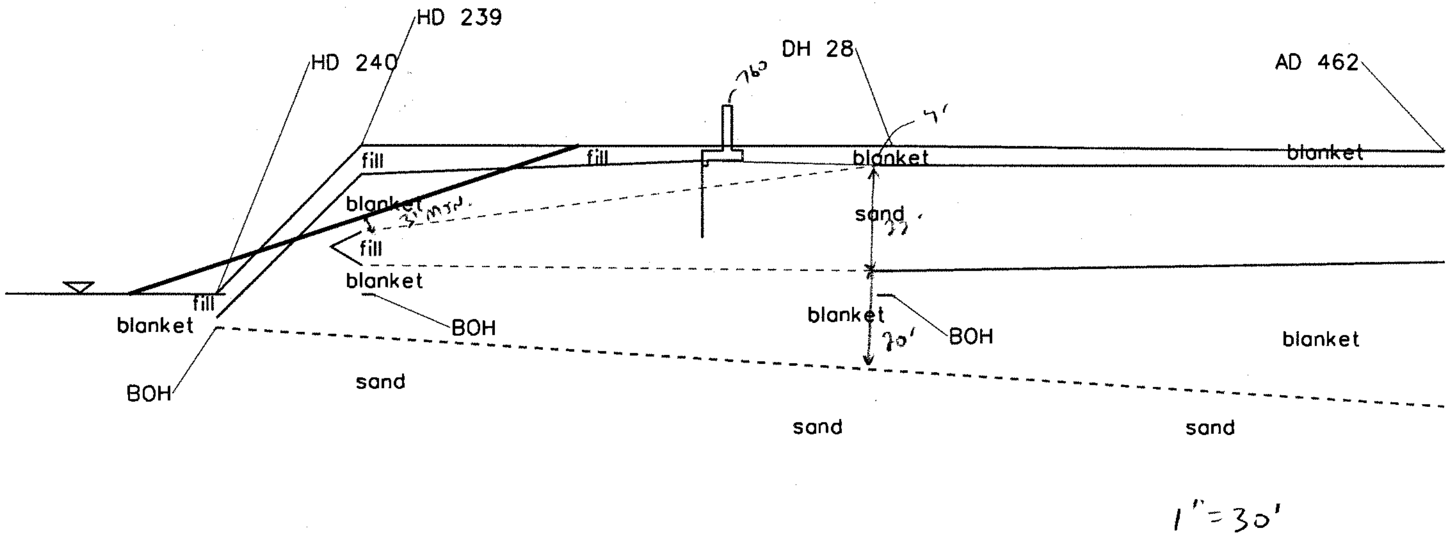
CID MO STATION 29+00



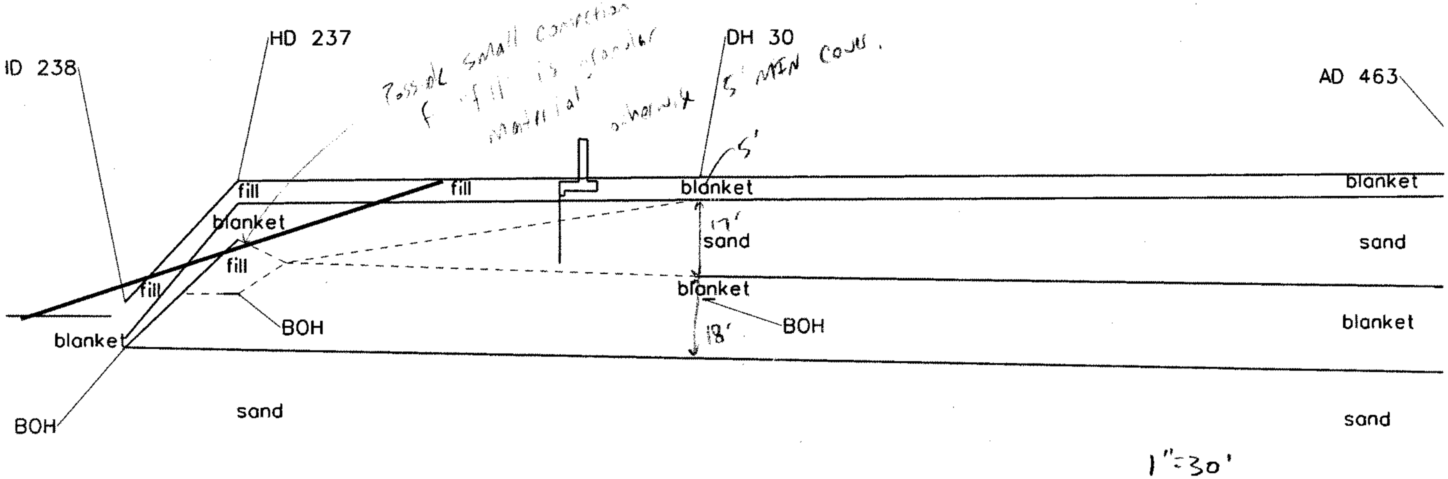
CID MO STATION 35+00



CID MO STATION 39+00



CID MO STATION 42+00



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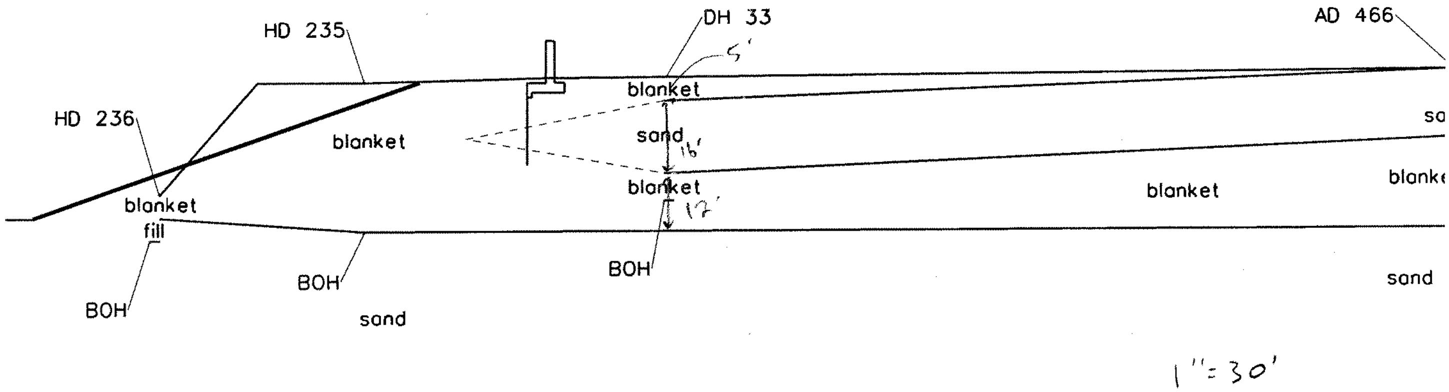
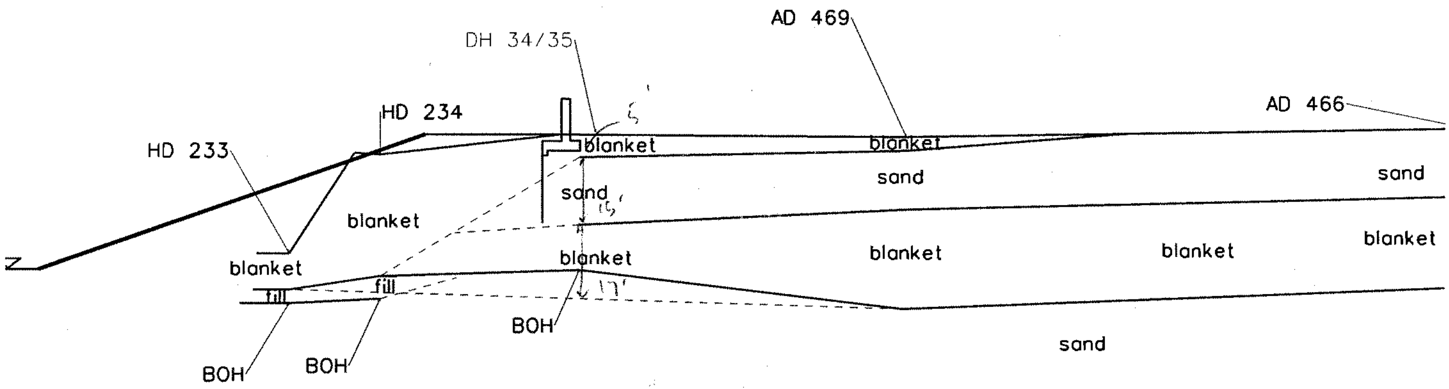
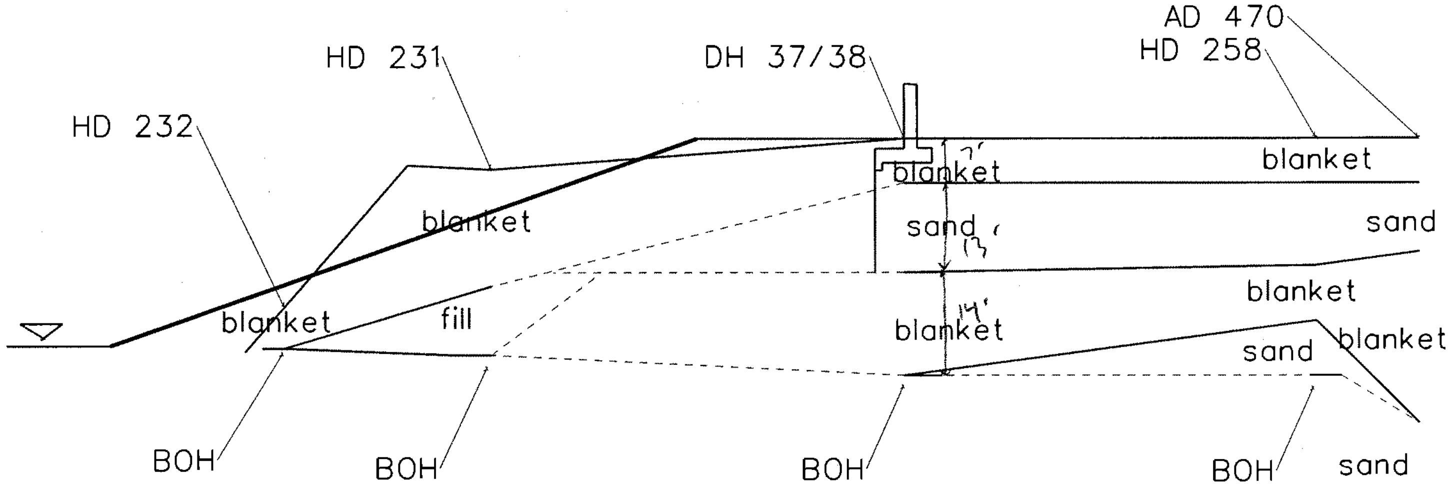


CID MO STATION 51+00



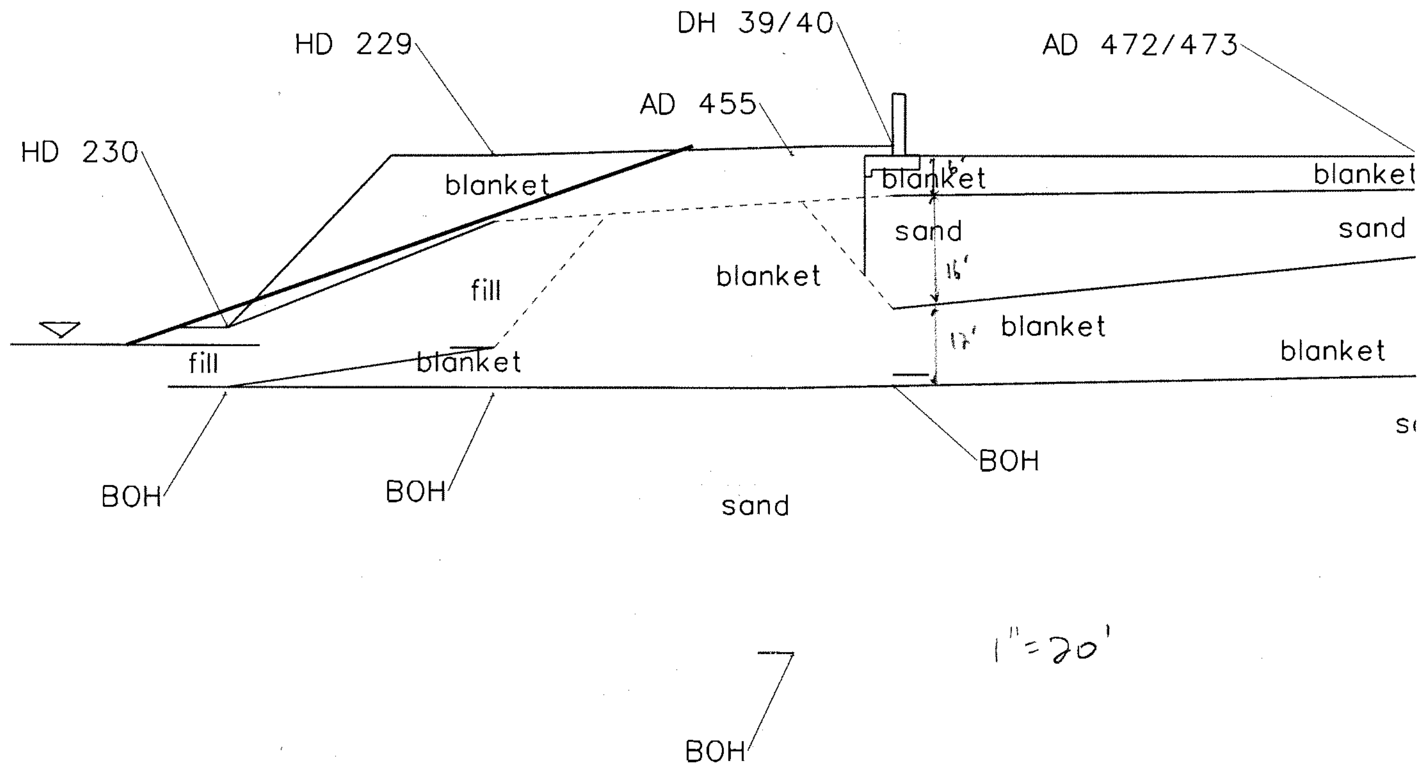
1" = 30'

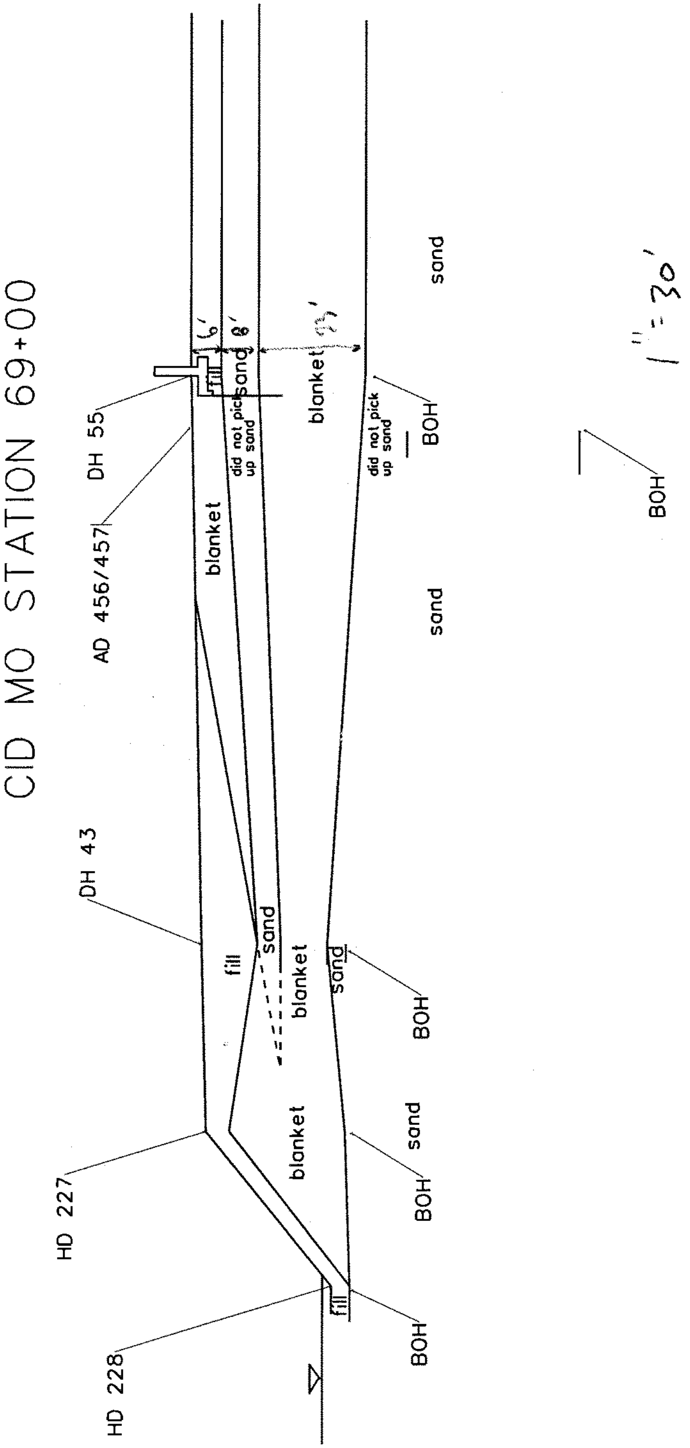
CID MO STATION 57+00



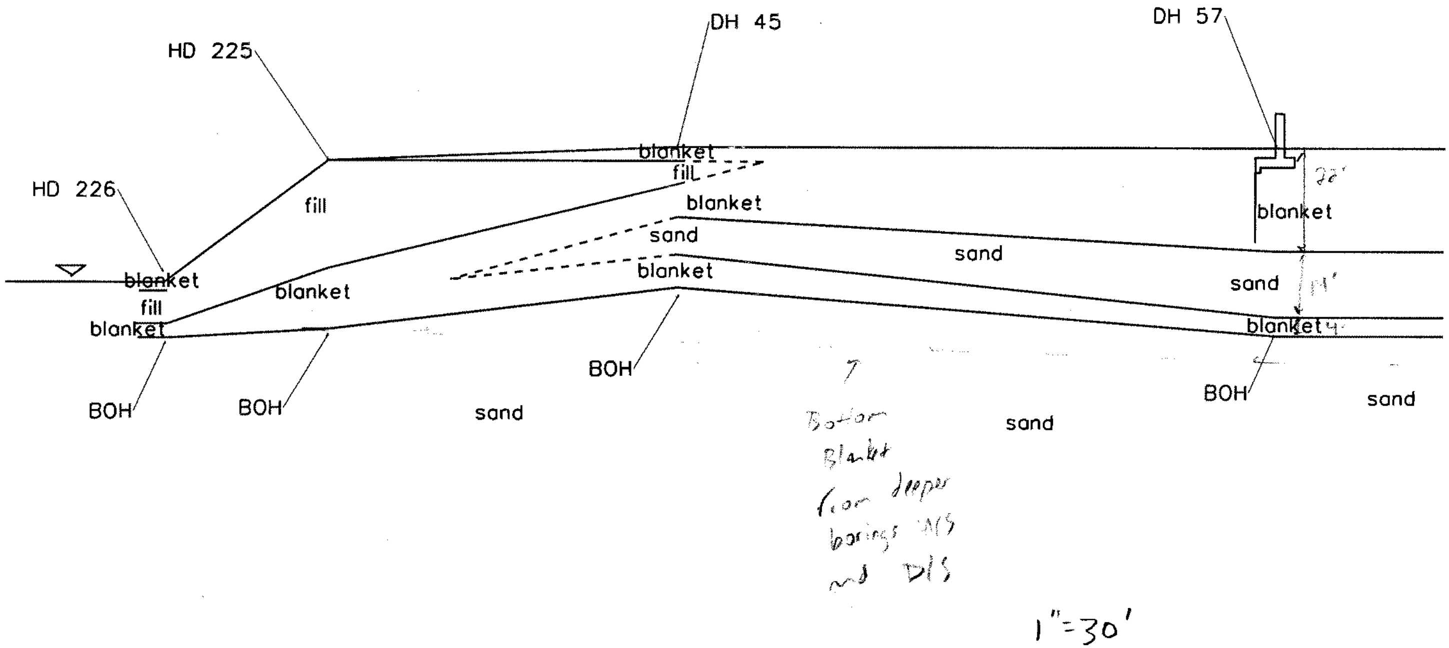
1" = 20'

CID MO STATION 62+00

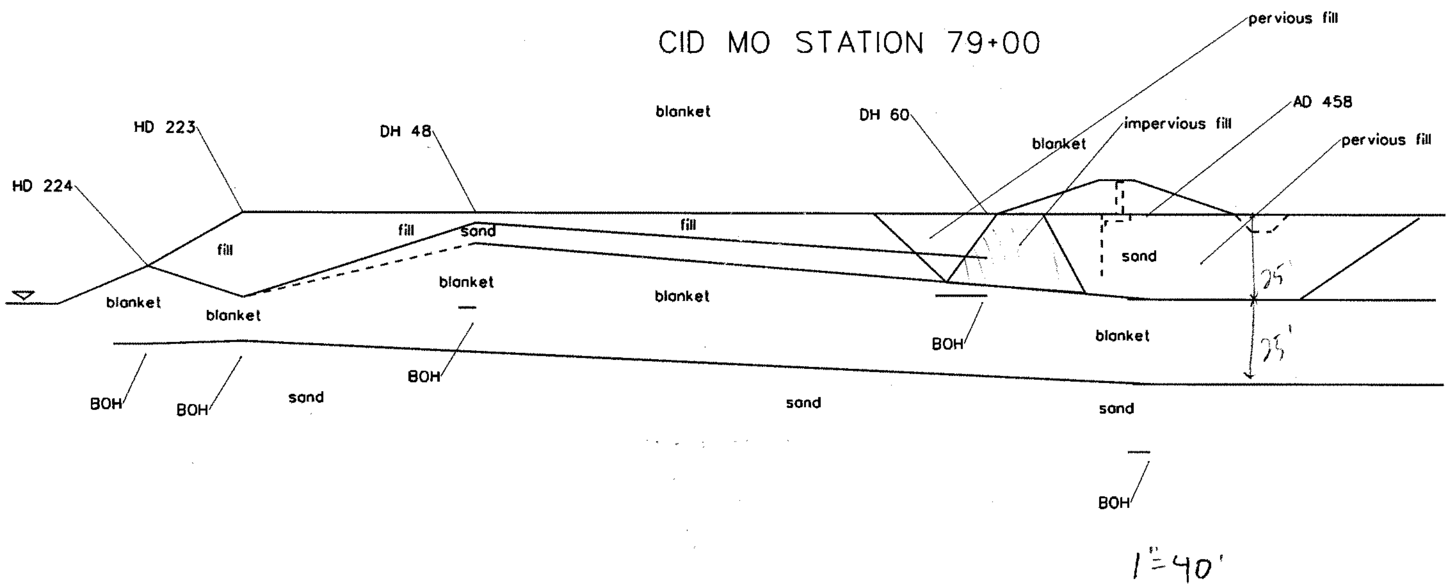



CID MO STATION 69+00
AD 456/457
DH 55
DH 43
HD 227
HD 228
6'
8'
13'
fill
sand
blanket
did not pick up sand
BOH
1" = 30'

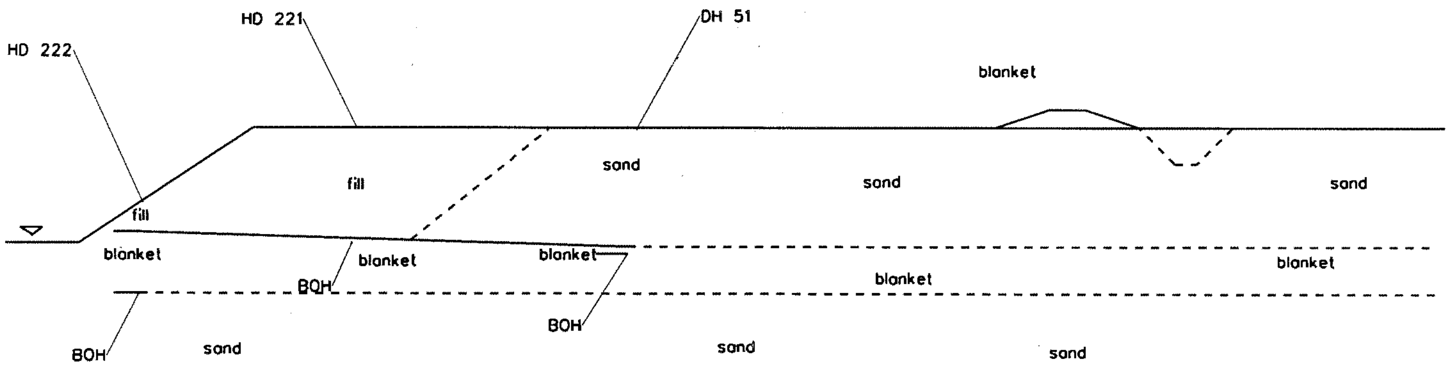
CID MO STATION 73+00



CID MO STATION 79+00



CID MO STATION 86+00



1"=40'

CID MO STATION 89+00

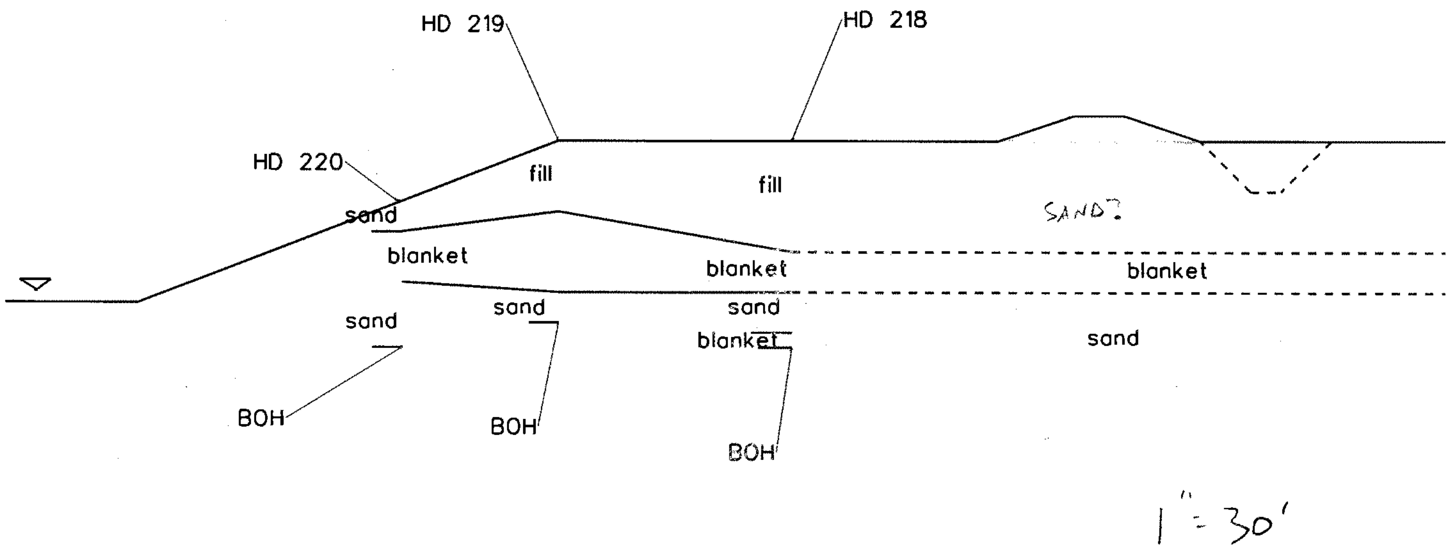


EXHIBIT A-5.2

NWK Levee Underseepage Guidance

CENWK-ED-GD 28 February 2011

MEMORANDUM for RECORD

SUBJECT: Kansas City District Levee Underseepage Guidelines

1. PURPOSE

This memorandum documents levee underseepage guidelines that Kansas City District (NWK) will use until updated USACE underseepage guidelines are available. When USACE guidelines are revised, NWK guidelines will be reviewed and revised if necessary.

The underseepage guidelines will be applied to NWK projects currently in Pre-Construction, Engineering, and Design (PED) phase. Current PED project details and consequences of failure are listed in Table 1. Project details were obtained from completed feasibility reports for the respective projects. The NWK underseepage guidelines will also be used in other NWK PED and feasibility study projects with similar failure consequences that are not currently planned. If NWK undertakes a future PED or feasibility study project that has lower failure consequences, NWK guidance will be developed for those projects at the beginning of the engineering effort in a Memorandum for Record.

Table 1 Current NWK Projects

Project	Nominal Frequency of Overtopping[3]	Economic Damages of Failure	Population at Risk	Failure Consequences
Topeka Oakland Unit	~300 yr	$578 million	7,600[1]	Very High
North Topeka Unit	~300 yr	$1.47 billion	8,200[1]	Very High
North Kansas City Unit	~750 yr	$3 billion	31,585[2]	Very High
MRLS L-455	~500 yr	$1.43 billion	3,700[1]	Very High
MRLS R-471-460	~200 yr	$571 million	2,000[1]	Very High

1 – Assumes 2.5 persons per residential and non-residential structure
2 – Residential population and employment population.
3 – Nominal frequency of overtopping is equivalent to the 50% confidence level, or expected value.

2. BACKGROUND

U.S. Army Corps of Engineers (USACE) underseepage design guidelines are being revised. Current USACE guidelines are contained in Engineer Manual (EM) 1110-2-1913 *Design and Construction of Levees*, 30 April 2000. New underseepage guidelines superseding many of the recommendations in EM 1110-2-1913 were published in Engineer Technical Letter (ETL) 1110-2-569 *Design Guidance for Levee Underseepage*, 1 May 2005. ETL 1110-2-569 stated that it would be rescinded when EM 1110-2-1913 was revised. EM 1110-2-1913 has not been revised but the ETL officially expired in May 2010. A draft version of revised EM 1110-2-1913, Appendix C - *Design of Seepage Berms* was issued for USACE internal review in October 2006 but has not been finalized. EC 1110-2-6067 *USACE Process for the National Flood Insurance*

Program (NFIP) Levee System Evaluation, 31 August 2010, states expired ETL 1110-2-569 should be used as a guide to evaluate levee structures and is current USACE policy instead of the requirements in the current EM 1110-2-1913.

Some USACE districts have been establishing local underseepage guidelines based on current, expired, and draft USACE underseepage guidelines. For example, the New Orleans Hurricane and Storm Damage Risk Reduction System (HSDRSS) guidelines in use by New Orleans District (MVO) were presented at the Geotechnical and Materials Community of Practice (CoP) Meeting in St. Louis in August 2010. In this memorandum, NWK is establishing underseepage guidelines for district use.

3. COMPARISON OF USACE AND NWK UNDERSEEPAGE GUIDELINES

There have been significant variations between current, recently expired, and draft USACE underseepage guidelines and historical NWK practice. The proposed NWK underseepage guidelines simplify and standardize underseepage guidelines for design and analysis of levees and floodwalls. NWK-proposed underseepage guidelines are based on an adequate subsurface exploration being performed to have a high confidence in the blanket thickness in accordance with standard engineering practice and published USACE guidelines. The general requirement in ETL 1110-2-569 is three borings (landside toe, riverside toe, and levee crest) every 1,000 feet, supplemented where appropriate with geophysical investigation. The subsurface investigation guidance in ETL 1110-2-569 will be considered the absolute minimum investigation required, and will typically be surpassed.

EM 1110-2-1913 states that the net head on a levee for gradient calculations is usually based on the "design water surface" elevation but is sometimes based on the top of levee (p. B-3). The "design water surface" is defined as the design or project flood stage, or the top of levee minus the freeboard allowance (typically 2 to 3 feet). In ETL 1110-2-569 water elevation recommendations for gradient calculations are not given, indicating the guidelines in EM 1110-2-1913 are still applicable. In Draft Appendix C, EM 1110-2-1913 the water elevation for gradient calculations is based on water at the top of levee or at the elevation of 100, 250, or 500–year frequency flood events based on consequences of failure. NWK traditional methodology checked gradient factor of safety (FS_i) with water at the design water surface and at the top of levee. The current proposed NWK underseepage guidelines standardize all calculations to a water elevation equal to the design top of levee elevation, excluding overbuild.

The following tables show the underseepage recommendations in current, expired, and draft USACE publications, traditional NWK guidelines, and proposed NWK guidelines.

A comparison of underseepage guidelines at the levee landside toe is shown in Table 2. One of the largest evolutions in underseepage guidelines has been from an exit gradient check to a FS_i check. This change was made because the critical gradient is variable with blanket unit weight. Having a check based on exit gradient meant that FS_i was variable for different soil types. Other changes have been proposed to change the design water loading and to take into account consequences of failure and flood frequency. The NWK guidelines for FS_i at the levee toe were developed to generally align with ETL 1110-2-569 requirements, which imply a FS_i of 1.6 is desired. The guidelines in Draft EM 1110-2-1913 are not well understood for very high failure consequence levees. It is not clear if levees with a higher frequency of overtopping are intended to be designed to a higher FS_i, or if the intent is to check FS_i at less than extreme loading conditions. Regardless of the interpretation of Draft EM 1110-2-1913, NWK considers the FS_i guidelines recommended in ETL 1110-2-569 appropriate for very high consequence levees for the extreme loading condition. The recommended FS_i of 1.6 in Table 2 will apply to the landside levee toe for all circumstances; whether the natural landside blanket, underseepage berms, relief wells, or other features are providing underseepage protection. When relief wells are used, the recommended FS_i will be met at all points in between relief wells.

Table 2 Underseepage Guidelines at Levee Landside Toe

<table>
<tr><th>Document</th><th>Current EM 1110-2-1913, April 2000</th><th>Expired ETL 1110-2-569, May 2005</th><th colspan="3">Draft EM 1110-2-1913 Appendix C, October 2006</th><th>Traditional NWK Guidelines</th><th>NWK Guidelines, 2011</th></tr>
<tr><td rowspan="5">Underseepage Guidelines - Levee Landside Toe</td><td rowspan="5">Gradient through cohesive blanket < 0.8 evaluation <0.3 new seepage control</td><td rowspan="5">Gradient through cohesive blanket < 0.5</td><td colspan="3">Consequences</td><td rowspan="5">FS_i = 1.1, water at levee top and FS_i = 1.5, design water surface</td><td rowspan="5">FS_i = 1.6, water at levee top</td></tr>
<tr><td>Low</td><td>High</td><td>Very High</td></tr>
<tr><td rowspan="3">FS_i = 1.3</td><td rowspan="3">FS_i = 1.6</td><td>FS_i = 2.0,100 yr frequency event</td></tr>
<tr><td>FS_i = 1.8, 250 yr frequency event</td></tr>
<tr><td>FS_i = 1.6, 500 yr frequency event</td></tr>
</table>

A comparison of underseepage guidelines at underseepage berm toes is shown in Tables 3a through 3c. Again, the largest evolution has been from a gradient check to a FS_i check. Other changes have been proposed to take into account confidence in design parameters and ratio of levee height and berm width. The NWK guidelines were developed to generally align with the guidelines in Draft EM 1110-2-1913. NWK agrees that reducing the minimum FS_i at berm toes as distance from the levee toe increases is a sound approach. This approach considers that failure risk is reduced as the distance from the levee increases. However, the NWK guidelines specify the recommended FS_i at berm toes with distance from the levee toe regardless of levee height. Specifying FS_i by distance from the levee toe simplifies the design process and removes unique situations of short height levees with high failure consequences having relatively low factors of safety close to the levee toe. The guidelines can be linearly interpolated for intermediate distances. For typical levee heights between 10 and 20 feet, the NWK guidelines generally meet or exceed the recommendations in Draft EM 1110-2-1913.

The recommended FS_i in Table 3c will also apply to other features landward of the levee where underseepage may be a concern. The recommended FS_i will apply to features such as interior drainage ditches, localized depressions, pits, or any other feature landward of the levee toe. The recommended FSi will apply whether the natural landside blanket or underseepage control measures are providing underseepage protection at the feature location.

A comparison of underseepage berm width, thickness, and overbuild guidelines is shown in Table 4. These guidelines have not changed significantly with evolving criteria. The NWK guidelines simplify the guidelines with the intent that FS_i will control designs instead of arbitrary minimum or maximum recommendations.

Table 3a Underseepage Guidelines at Underseepage Berm Toe

Document	Current EM 1110-2-1913, April 2000	Expired ETL 1110-2-569, May 2005	Traditional NWK Guidelines
Underseepage Guidelines - Levee Underseepage Berm Toe	Gradient through cohesive blanket < 0.8	Not addressed, use EM	FS_i = 1.1 for Urban Levees and FS_i = 0.8 for Agricultural Levees, water at design water surface

CENWK ED-GD
SUBJECT: Kansas City District Levee Underseepage Guidelines

Table 3b Underseepage Guidelines at Underseepage Berm Toe

Document	Draft EM 1110-2-1913 Appendix C, October 2006 – small uncertainty*, for consequences as shown below											
Underseepage Guidelines - Levee Underseepage Berm Toe	Berm Width/Levee Height ≤ 4			Berm Width/Levee Height < 8			Berm Width/Levee Height < 12			Berm Width/Levee Height < 16		
	Low	High	Very High	Low	High	Very High	Low	High	Very High	Low	High	Very High
	FS_i = 1.05	FS_i = 1.30	FS_i = 1.50	FS_i = 1.10	FS_i = 1.15	FS_i = 1.30	FS_i = 0.95	FS_i = 1.00	FS_i = 1.10	FS_i = 0.90	FS_i = 0.90	FS_i = 0.90

*based on obtaining adequate subsurface information for small uncertainty in design parameters; document also shows recommended FS_i for large uncertainty

Table 3c Underseepage Guidelines at Underseepage Berm Toe

Document	NWK Guidelines, 2011				
Underseepage Guidelines - Levee Underseepage Berm Toe	Distance From Landside levee Toe (ft)				
	100*	200	300	400	500+
	FS_i = 1.5	FS_i = 1.4	FS_i = 1.3	FS_i = 1.2	FS_i = 1.1

*100 feet is the proposed minimum berm width from the landside levee toe

Table 4 Underseepage Berm Width, Thickness, and Overbuild Guidelines

Document	Current EM 1110-2-1913, April 2000	Expired ETL 1110-2-569, May 2005	Draft EM 1110-2-1913 Appendix C, October 2006	Traditional NWK Guidelines	NWK Guidelines, 2011
Minimum Berm Width	150 feet	4 X Levee Height	4 X Levee Height	NA*	100 feet
Maximum Berm Width	400 feet	Use caution when limiting berm width to 300-400 feet	No Limit Specified	NA	No Limit, FS_i guidelines must be met
Minimum Berm Thickness – Levee Toe	5 feet	5 feet	5 feet	NA	5 feet
Minimum Berm Thickness – Berm Toe	2 feet	2 feet	2 feet	NA	2 feet
Berm Thickness Increase for Shrinkage and Consolidation	25%	Calculate based on Consolidation Theory	Calculate based on Consolidation Theory	NA	25% if no consolidation data available, or as calculated

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SUBJECT: Kansas City District Levee Underseepage Guidelines

					with consolidation data
Berm Slope for Surface Drainage	Generally 1V on 50H or steeper	NA	Generally 1V on 50H or steeper	Not Specified	Minimum 1 % away from levee for 100 ft, graded away from levee at all distances.

*Not addressed – Same as EM 1110-2-1913, April 2000

4. CONCLUSION

The NWK underseepage guidelines are intended to be realistically conservative, practical, and easy to implement. They were developed to generally satisfy the intent of recent USACE underseepage guidelines and provide standardized guidance for NWK. As always, the use of engineering judgment is recommended for each individual project. The Kansas City District will use these underseepage guidelines until updated general USACE guidance is available. Development of this memorandum was coordinated with NWD (Y. Rhee, S. Fink) through a series of phone calls and emails in January and February 2011. Additionally, limited coordination with HQUSACE (J. Koester) and ERDC (K. Klaus) was performed via email

David L. Mathews, PE
Chief, Geotechnical Branch

CENWK ED-GD
SUBJECT: Kansas City District Levee Underseepage Guidelines

JENSON
ED-GD

~~BELCEW~~
~~ED-GD~~

for VOLLINK gjh
ED-GD

JONES
ED-GD

7

EXHIBIT A-5.3

CID MO – Soil Strength Parameters

PEER RSK

CID-MO | STRENGTH PARAMETERS KCL2 FEASIBILITY 1/4

GMB

CID-MO UNIT

FROM 2001 SPT'S (SEE ATTACHED TABLE)

SAND $\bar{c} = 0$ $\bar{\phi} = 30°$

BLANKET $C_u = 600$ psf $\phi_u = 0$

$\bar{c} = ?$ $\bar{\phi} = ?$

CID-MO UNIT ADOPTED VALUES FOR FEASIBILITY

	γ_{moist} (pcf)	γ_{SAT} (pcf)	C_u (psf)	$\bar{c}$ (psf)	$\bar{\phi}$ (Deg)
EMBANKMENT :	115	120	1000	0	29°
BLANKET* :	110	115	600	0	22°
SAND :	115	120	N/A	0	30°
FILL/DEBRIS+ :	110	115	600	0	20°

* CH MATERIAL NOT INCLUDED.

\+ Assumed parameters based on weakest perceived material likely to be present.

ABOVE VALUES CHOSEN BASED ON AVAILABLE SITE SPECIFIC INFORMATION AND AVAILABLE ADJACENT INFORMATION INCLUDED HERE.

2/4

NORTH KANSAS CITY UNIT (ACROSS MO RIVER)

ANALYSIS OF DESIGN ... (MAY 1945)

BLANKET NO MATERIALS GIVEN

1) $\bar{c}$ = 0 psf $\bar{\phi}$ = 27° ML/SM
2) $\bar{c}$ = 500 psf $\bar{\phi}$ = 14° CH
3) $\bar{c}$ = 200 psf $\bar{\phi}$ = 22° CL
4) $\bar{c}$ = 0 psf $\bar{\phi}$ = 22° CL/ML

ASSUMED MATERIAL

FAIRFAX JERSEY CREEK (U/S on MO RIVER)

FOUND IN NKC DEFINITE PROJECT REPORT (DEC, 1943)

STABILITY ANALYSIS PLATE

SAND	$\bar{c}$ = 0	$\bar{\phi}$ = 31°
SANDY LOAM	$\bar{c}$ = 0	$\bar{\phi}$ = 31°
SILTY LOAM	$\bar{c}$ = 0	$\bar{\phi}$ = 29°
SILTY CLAY LOAM	$\bar{c}$ = 0	$\bar{\phi}$ = 22°
CLAY LOAM	$\bar{c}$ = 0	$\bar{\phi}$ = 24°
LEAN CLAY	$\bar{c}$ = 0	$\bar{\phi}$ = 17°
MEDIUM CLAY	$\bar{c}$ = 0	$\bar{\phi}$ = 17°

EAST BOTTOMS UNIT (D/S on MO RIVER)

ANALYSIS OF DESIGN TO ACCOMPANY CONSTRUCTION PLANS FOR LEVEE, FLOODWALL AND APPURTENANCES

SEPTEMBER 1945

DESIGN DRAINED STRENGTHS ON MO. RIVER REACH

	psf	Degrees
FAT CLAY	$\tau = 100$	$\bar{\phi}$: 12°
MEDIUM CLAY	$\tau = 0$	$\bar{\phi}$: 15°
LEAN CLAY	$\tau = 360$	$\bar{\phi}$: 16°
SILTY CLAY	$\tau = 0$	$\bar{\phi}$: 26°
SILT	$\tau = 0$	$\bar{\phi}$: 30°

BEARING CAPACITY = 2000 psf

CID-KS UNIT (U/S on KS RIVER)

1962 MODIFICATION DM (Apr 1973)

FAT CLAY	$\tau = 400$ psf	$\bar{\phi}$: 17°
LEAN CLAY	$\tau = 0$	$\bar{\phi}$: 26°
SILT	$\tau = 0$	$\bar{\phi}$: 24°

ANALYSIS OF DESIGN ... (Feb 1946)

LEAN CLAY	$\tau = 100$	$\bar{\phi}$: 24°

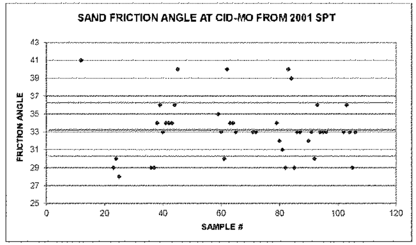
2007 FEASIBILITY STUDY

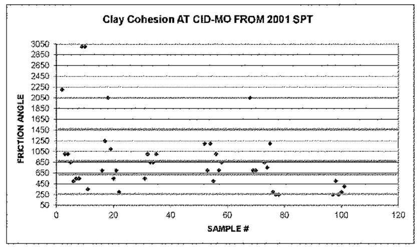
BLANKET	$\tau = 0$	$\bar{\phi}$: 24°
FILL	$\tau = 0$	$\bar{\phi}$: 21°
FNDN SAND	$\tau = 0$	$\bar{\phi}$: 31°

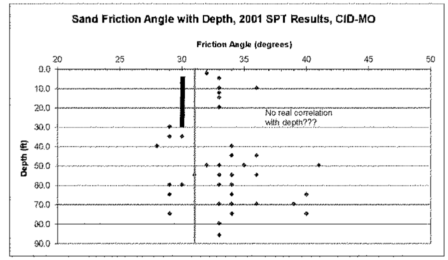
Kansas City Levees - CID MO
SPT Blowcounts in the aquifer - Drilling performed in October 2001
Data used to estimate shear strength of the foundation materials 2428.5 1214.25

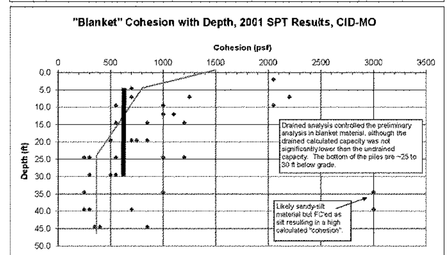
Moist Unit Weight = 115 pcf
Saturated Unit Weight = 120 pcf

Boring	Station	Location	Depth (ft)	Material (FC)	Location of Water Table (interpreted) (ft)	Overburden Pressure (tsf)	Overburden Correction C_n	Uncorrected Blowcount	Corrected Blowcount	Friction Angle φ' (degrees)	Undrained Shear Strength, Cu (psf)
D-454	29+75	12' RS	2.0	Fill (cinders, gravel)	11	0.12	1.79	22	39		
D-454	29+75	12' RS	4.5	Clayey Sand	11	0.26	1.59	14	22		
D-454	29+75	12' RS	7.0	CH	11	0.40	1.43	12	17		2200
D-454	29+75	12' RS	9.5	CH	11	0.55	1.29	8	6		1000
D-454	29+75	12' RS	12.0	CH	11	0.66	1.20	7	8		1000
D-454	29+75	12' RS	14.5	CH	11	0.73	1.15	6	7		850
D-454	29+75	12' RS	19.5	CH	11	0.88	1.07	4	4		500
D-454	29+75	12' RS	24.5	CH	11	1.02	0.99	5	5		550
D-454	29+75	12' RS	29.5	CH	11	1.17	0.92	5	5		550
D-454	29+75	12' RS	34.5	ML	11	1.31	0.87	28	24		3000
D-454	29+75	12' RS	39.5	ML	11	1.45	0.82	25	20		3000
D-454	29+75	12' RS	44.5	ML	11	1.60	0.77	3	2		350
D-454	29+75	12' RS	49.5	Sand	11	1.74	0.73	70	51	41	
			51.5	Rock							
AD-453	4+65	10' LS	2.0	Clayey Gravel w/sand	100	0.12	1.79	12	22		
AD-453	4+65	10' LS	4.5	CL w/sand	100	0.26	1.59	4	6		700
AD-453	4+65	10' LS	7.0	CL w/sand	100	0.40	1.43	9	13		1250
AD-453	4+65	10' LS	9.5	CH w/sand	100	0.55	1.29	12	16		2050
AD-453	4+65	10' LS	12.0	CH	100	0.69	1.18	8	9		1100
AD-453	4+65	10' LS	14.5	CH w/silt,sand	100	0.83	1.09	5	5		550
AD-453	4+65	10' LS	19.5	CH w/silt,sand	100	1.12	0.94	6	6		700
AD-453	4+65	10' LS	24.5	CH w/silt	100	1.41	0.83	4	3		300
AD-453	4+65	10' LS	29.5	Clayey Sand	100	1.70	0.74	8	6	29	
AD-453	4+65	10' LS	34.5	Clayey Sand	100	1.98	0.67	11	7	30	
AD-453	4+65	10' LS	39.5	Clayey Sand	100	2.27	0.61	4	2	28	
			43.7	Rock							
AD-455	63+00	15' RS	2.0	CL (sand, fill?)	24	0.12	1.79	12	22		
AD-455	63+00	15' RS	4.5	Sand w/silt	24	0.26	1.59	14	22		
AD-455	63+00	15' RS	7.0	Sand w/silt	24	0.40	1.43	11	16		
AD-455	63+00	15' RS	9.5	CH	24	0.55	1.29	4	5		550
AD-455	63+00	15' RS	12.0	OL? (CH/CL)	24	0.69	1.18	7	8		1000
AD-455	63+00	15' RS	14.5	CH w/ sand	24	0.83	1.09	6	7		850
AD-455	63+00	15' RS	19.5	Sandy CH	24	1.12	0.94	7	7		850
AD-455	63+00	15' RS	24.5	Sandy CH	24	1.39	0.84	9	8		1000
AD-455	63+00	15' RS	29.5	Sand	24	1.54	0.79	8	6	29	
AD-455	63+00	15' RS	34.5	Sand	24	1.68	0.75	8	6	29	
AD-455	63+00	15' RS	39.5	Sand	24	1.83	0.71	25	18	34	
AD-455	63+00	15' RS	44.5	Sand	24	1.97	0.67	34	23	36	
AD-455	63+00	15' RS	49.5	Sand	24	2.11	0.64	17	11	33	
AD-455	63+00	15' RS	54.5	Sand	24	2.26	0.61	30	18	34	
AD-455	63+00	15' RS	59.5	Sand	24	2.40	0.59	29	17	34	
AD-455	63+00	15' RS	64.5	Sand	24	2.55	0.56	34	19	34	
AD-455	63+00	15' RS	69.5	Sand	24	2.69	0.54	41	22	36	
AD-455	63+00	15' RS	74.5	Sand	24	2.83	0.52	70	37	40	
D-456	68+65	12' RS	4.5	Sandy Clay	34	0.26	1.59	18	29		
D-456	68+65	12' RS	7.0	Ashpalt/Gravel (fill)	34	0.40	1.43	10	14		
D-456	68+65	12' RS	9.5	Cement/Brick (fill)	34	0.55	1.29	7	9		
D-456	68+65	12' RS	12.0	Clay w/ cind. brick	34	0.69	1.18	6	7		
D-456	68+65	12' RS	14.5	Sandy Silt	34	0.83	1.09	11	12		1200
D-456	68+65	12' RS	19.5	Clayey Silt	34	1.12	0.94	6	6		700
D-456	68+65	12' RS	24.5	Clayey Silt	34	1.41	0.83	12	10		1200
D-456	68+65	12' RS	29.5	Clayey Silt	34	1.70	0.74	5	4		500
D-456	68+65	12' RS	34.5	Clayey Silt	34	1.97	0.67	12	8		1000
D-456	68+65	12' RS	39.5	Silty Clay	34	2.11	0.64	9	6		700
D-456	68+65	12' RS	44.5	Sandy Silt	34	2.26	0.61	12	7		850
D-456	68+65	12' RS	49.5	Silty Sand	34	2.40	0.59	36	21	35	
D-456	68+65	12' RS	54.5	Sand	34	2.55	0.56	28	16	33	
D-456	68+65	12' RS	59.5	Sand	34	2.69	0.54	12	7	30	
D-456	68+65	12' RS	64.5	Sand	34	2.83	0.52	100	52	40	
D-456	68+65	12' RS	69.5	Sand	34	2.98	0.50	37	19	34	
D-456	68+65	12' RS	74.5	Sand	34	3.12	0.49	28	14	34	
D-456	68+65	12' RS	79.5	Sand	34	3.27	0.47	23	11	33	
			82.5	Rock							
D-457	70+60	12' RS	2.0	CL	13.5	0.12	1.79	9	16		2050
D-457	70+60	12' RS	4.5	CH	13.5	0.26	1.59	4	6		700
D-457	70+60	12' RS	7.0	Sandy CH	13.5	0.40	1.43	4	6		700
D-457	70+60	12' RS	9.5	Sand	13.5	0.55	1.29	12	16	33	
D-457	70+60	12' RS	12.0	Sand	13.5	0.69	1.18	11	13	33	
D-457	70+60	12' RS	14.5	CH	13.5	0.81	1.11	6	7		850
D-457	70+60	12' RS	19.5	Clayey Silt	13.5	0.95	1.03	6	6		750
D-457	70+60	12' RS	24.5	Sand	13.5	1.09	0.96	10	10		1200
D-457	70+60	12' RS	29.5	CH	13.5	1.24	0.89	3	3		300
D-457	70+60	12' RS	34.5	Silt	13.5	1.38	0.84	0	0		250
D-457	70+60	12' RS	39.5	Silt	13.5	1.53	0.79	1	1		250
D-457	70+60	12' RS	44.5	Sand	13.5	1.67	0.75	24	18	34	
D-457	70+60	12' RS	49.5	Sand	13.5	1.81	0.71	20	14	32	
D-457	70+60	12' RS	54.5	Sand	13.5	1.96	0.68	14	9	31	
D-457	70+60	12' RS	59.5	Sand	13.5	2.10	0.64	9	6	29	
D-457	70+60	12' RS	64.5	Sand	13.5	2.25	0.62	66	41	40	
D-457	70+60	12' RS	69.5	Sand	13.5	2.39	0.59	58	34	39	
D-457	70+60	12' RS	74.5	Gravel/Sand	13.5	2.53	0.57	11	6	29	
D-457	70+60	12' RS	79.5	Gravel/Sand	13.5	2.68	0.54	23	13	33	
D-457	70+60	12' RS	85.5	Gravel/Sand	13.5	2.85	0.52	27	14	33	
			87.0	Rock							









Boring	Station	Location	Depth (ft)	Material (FC)	Location of Water Table (interpreted) (ft)	Overburden Pressure (tsf)	Overburden Correction C_n	Uncorrected Blowcount	Corrected Blowcount	Friction Angle φ' (degrees)	Undrained Shear Strength, Cu (psf)
D-458	80+30	10' LS	2.0	Sand	11	0.12	1.79	12	22	32	
D-458	80+30	10' LS	4.5	Sand	11	0.26	1.59	7	11	33	
D-458	80+30	10' LS	7.0	Gravel	11	0.40	1.43	9	13	30	
D-458	80+30	10' LS	9.5	Sand	11	0.55	1.29	17	22	36	
D-458	80+30	10' LS	12.0	Sand	11	0.66	1.20	13	16	33	
D-458	80+30	10' LS	14.5	Sand	11	0.73	1.15	10	12	33	
D-458	80+30	10' LS	19.5	Sand	11	0.88	1.07	15	16	33	
D-458	80+30	10' LS	24.5	Sandy CH	11	1.02	0.99	2	2		250
D-458	80+30	10' LS	29.5	CH	11	1.17	0.92	4	4		500
D-458	80+30	10' LS	34.5	CH	11	1.31	0.87	0	0		250
D-458	80+30	10' LS	39.5	CH	11	1.45	0.82	4	3		300
D-458	80+30	10' LS	44.5	CH	11	1.60	0.77	3	2		400
D-458	80+30	10' LS	49.5	Clayey Sand	11	1.74	0.73	15	11	33	
D-458	80+30	10' LS	54.5	Sand	11	1.89	0.69	31	21	36	
D-458	80+30	10' LS	59.5	Sand	11	2.03	0.66	18	12	33	
D-458	80+30	10' LS	64.5	Sand	11	2.17	0.63	12	8	29	
D-458	80+30	10' LS	69.5	Sandy CH	11	2.32	0.60	19	11	33	

Notes:

1. Overburden correction to blowcount: $C_n = 2/(1 + \sigma')$, with σ' in tsf
2. Corrected blowcount = $N_{(uncorrected)} * C_n$, safety hammer on cat head rig, so no other correction per EM
3. C_u from EM 1110-1-1804 Table G-1 and EM 1110-2-2504 Table 3-4
4. φ from EM 1110-1-1905 Table 3-1 and EM 1110-2-2504 Table 3-1

	φ' (degrees) AVG	φ' (degrees) STDEV	C_u (psf) AVG	C_u (psf) STDEV
Sand	33.3	3		
CL			983	472
CH			763.4615385	478
Silt			1283	1348
blanket			902	658

Material	Strength		Source
C_u blanket =	600	psf	SPT
φ blanket =	N/A	degrees	SPT
φ sand =	30	degrees	SPT
φ fill =	N/A	degrees	SPT

EXHIBIT A-5.4

CID MO – Shallow Foundation Capacity

STA 0+00 → 7+00

SHALLOW FOOTING @ el 745 GROUND @ el 752.

BEARING IN BLANKET MATERIAL @ D_F = 6 ft

q_{ult} = 3400 psf ✓

STA 7+00 → 7+50

SHALLOW FOOTING @ el 740 GROUND @ el 750.

BEARING IN BLANKET MATERIAL @ D_F = 10 ft

q_{ult} = 3610 psf

STA 7+50 → 10+25

SHALLOW FOOTING @ el 736 GROUND @ el 747

BEARING IN BLANKET MATERIAL @ D_F = 11

q_{ult} = 3663 psf ✓

STA 10+25 → 22+31.46

SHALLOW FOOTING BEARING ON ROCK. (LIMESTONE)

q_{all} = 20,000 psf

13 JULY 1999 MFR (Geology) Geological Report for Design of Brush Creek-Paseo Extension Contract Modification, Kansas City, MO (ATTACHED)

Assume $q_{ult} = q_{all} * \overset{F.S.}{2}$

q_{ult} = 40,000 psf

AASHTO HB-17
TABLE 4.11.4.1.4-1

EXHIBIT A-5.5

Ultimate Design Pile Capacity Calculations and Summary

Summary of Calculated **Drained Ultimate Design Pile Capacities**, CID-MO Unit

Station Start	Station Stop	Pile Size, inches	Pile Shape	Pile Length, feet	Axial Compressive Capacity, lbs	Axial Tensile Capacity, lbs
22+81.46	24+54.76	18	tapered	21	**63,184**	34,428
24+54.76	25+38.76	16	straight	25	**94,544**	47,066
25+38.76	26+22.76	16	straight	29	**117,871**	60,583
26+22.76	27+90.76	16	straight	30	**110,124**	56,280
22+81.46	27+90.76	10	cut off pile*	16	22,839	**9,174**
27+90.76	30+42.75	16	straight	34	**165,730**	93,113
30+42.75	32+52.76	16	straight	21	**87,472**	44,990
27+90.76	30+42.75	10	cut off pile*	16	34,658	**7,421**
32+52.76	48+06.76	16	straight	21	**118,051**	67,168
32+52.76	48+06.76	10	cut off pile*	16	46,675	**15,817**
48+06.76	60+24.76	18	tapered	21	**76,858**	45,490
48+06.76	60+24.76	10	cut off pile*	16	28,969	**14,351**
60+24.76	73+20.14	18	straight	34	**137,963**	73,264
73+20.14	78+12.22	16	straight	21	**67,405**	34,166
60+24.76	78+12.22	10	cut off pile*	16	20,730	**9,401**

*cut off pile capacities are in lb/ft

CALCULATED BY GLEN BELLEW
PEER REVIEWED BY SCOTT LOEHR ON AUGUST 25, 2009 and SEPTEMBER 01, 2011

08/27/09

CID-MO "DESIGN"
ULTIMATE PILE CAPACITIES

CALC: GLEN M BELLEW, PE
PEER: SCOTT A LOEHR, PE

SCAN to File the CALCS

Peer Review
Documentation

Subject: Peer Review of Central Industrial District – MO, Geotechnical Pile Capacities
Date: August 25, 2009

Responses by Glen M Bellew (GMB), 08/26/09

1. The subject project information provided included as built drawings, soils characterization along the floodwall profile, underseepage calculations, and hand calculations for determining the ultimate capacities of the existing floodwall pile and concrete cutoff wall.

GMB – Noted.

2. The data presented did not provide any boring information. It is assumed that all available data was reviewed including the 2001 boring taken to obtain SPT blow counts for some reaches of the floodwall, at eh pump plants and at the floodwall gaps. The technical documentation should include a discussion of the available data used for the assessment.

GMB – Boring information was obtained from the "as-built" drawings, the 2001 borings, and some additional 2009 borings. This information will be provided and discussed in the technical documentation for the analysis.

3. The profiles provided did not shown the depth to which the piles were founded for the general reaches identified in the analyses. The spread footing versus pile foundation would be helpful for future reviews. Even of the piles are at a constant depth future documentation illustrations should assure that a note or a market is shown on the profiles.

GMB – the profiles were developed for an underseepage analysis and were subsequently used for the pile capacity analysis. The length of each driven pile for the entire CID-MO floodwall was provided on the "as-built" drawings. The elevation of the pile cap was also provided on the "as-built" drawings. From the information provided, the elevations of the pile top and bottom can be obtained. In light of the information available, the pile depths will not be shown on the profile.

4. During review of the calculations provided, the reviewer assumes this package is to support the development of the existing conditions (EC) assessment. The following comments are related to this assumption:

a. For an EC assessment, the soil parameters appear to be on the conservative side of the expected mean values. If using for design use – the parameters appear acceptable. If for an EC condition, the use of a low phi angle yields a very low value for the Nq values used for end bearing. Minor variations in phi will yield considerable increases in the Nq and end bearing resistance capacity. No calculations are shown to consider the variations in the expected soils parameters and resultant variations in the Qult.

GMB – The analysis provided was for a "design" ultimate capacity of the existing condition. These "design" values will be used by ED-DS to calculate a factor of safety for the piles. If any areas have low factors of safety under "design" conditions, a reliability analysis will be performed for those areas. For a reliability analysis of the existing conditions, an "expected" or "mean" ultimate pile capacity will be provided (considering a slightly higher strength as Reviewer stated) in the weak areas, and that ultimate pile capacity will be varied in accordance with the values provided in ETL 1110-2-561.

b. If for an EC assessment, the use of the EM and the limiting critical depth that truncated the effective overburden has been questioned by the District in the past during the development of Qult for Fairfax. I believe the critical depth was implemented in the FHWA design critical by Dr. O'Neil after a directive by the FHWA and the Corps also adopted it for DESIGN purposes. If the EC truly utilizes all available resistance with depth, the total effective overburden pressure should be utilized. The reality of a critical depth has been questioned by Dr. Kulhawy of Cornell, Dr. Duncan of Virginia Tech and other at ER-DC. ITR reviewer may question NWK on the use of the D_c.

GMB – You are correct that the limiting critical depth does not appear to be very highly regarded by most people anymore. The FHWA manual from 1998 actually specifies to use no limiting values of side friction or bearing resistance for effective stress analysis. Other literature specifies to use a limiting value (but not related to only pile diameter), but it only comes into play at depths greater than what we have here. The calculations were revised to include no limits on side friction or bearing resistance for the effective stress analysis. This increased the pile capacities slightly in those reaches where the pile was longer than the previously calculated critical depth.

c. The use of one of 4 sets of underseepage pressures assumptions needs to be documented. If the rationale is conservative for design reasons, that is easy to understand. But using one or the other for EC does not incorporate the variability of the potential changes in the foundations pressures due to the presence of highly variable layering. The assessment should address the selection of assumption 3 for the upper sands and assumption 4 for the lower sands.

GMB – Assumptions 1 and 2 represent an absolute best case and an absolute worst case with respect to underseepage. Assumption 1 is fairly realistic, but is probably overconservative. Assumption 2 is not really feasible, but is just a lower bound solution.. Assumptions 3 and 4 are considered to best represent the underseepage characteristics of the foundation at CID-MO and were therefore used in the pile capacity calculations.

5. The final documentation of the capacities should identify the meaning of the values in terms of a representation of a conservative design value or for use as an expected mean values for EC report. If the intent of this assessment is for providing conservation design values for structural use in assessing the future condition need for the Central Industrial District - MO, the calculations appear to have adequately used the EM guidance requirements to provide a geotechnical recommendation that is reasonable.

GMB – the capacities will be documented as" design ultimate capacities" when they are provided to ED-DS. However, they do not represent "conservative design" values, but simply "design" values. There is always some inherent conservatism in "design" values, and certainly there is some inherent conservatism in the calculated pile capacities. But I do not believe there is evidence of excess conservatism that it is deemed noteworthy to call the capacities "conservative" design values.

6. If you have any question please contact Scott Loehr at 816-389-3601.

Scott A. Loehr
Scott Loehr, P.E.
Geotechnical Engineer

7. Reviewed revision made to hand calculations to represent the expected value soil mechanic parameters. Some math corrections noted and comments provided. Scott Loehr 9/1/2011

Pile Capacity Calculations

Revised.

Drained Revised

Undrained N/A

Is a value of 22° representative of an expected value for blanket - could be raised somewhat? it could be a little low but not unreasonably so.

CID-MO Pile FNDN Axial Cap. 08/20/09 BELLEW 1/

EM 1110-2-2906 Design of Pile Foundations, Method.

Revised 08/26/09

Drained Revised 09/01/11

Pile Capacity, $Q_{ult} = Q_s + Q_t$

side friction capacity, $Q_s = f_s A_s$

tip capacity. $Q_t = q A_t$

\# EM suggests limiting f_s and q to a critical depth related to pile diameter for effective stress analysis. However, FHWA and others say that the critical depth concept is ill advised. The general consensus is to use a limiting fs and q based on material type and pile type, but it only comes into play at great depths therefore, no limiting factors will be used here in the Revised Analysis

Undrained Analysis

$$Q_s = \overbrace{\alpha C}^{f_s} A_s$$

$$Q_t = \underbrace{9C}_{q} A_t$$

C = cohesion or undrained shear strength

A_s = Area in contact with soil

α = Adhesion factor = 0.9 from Fig 4-5a

Drained Analysis

$$Q_s = \underbrace{\bar{\sigma}_v K \tan \delta}_{f_s} A_s$$

$\bar{\sigma}_v$ limited to $\bar{\sigma}_{v_{D_c}}$

$$Q_t = \bar{\sigma}_{v_t} N_q A_t$$

$\bar{\sigma}_{v_t}$ limited to $\bar{\sigma}_{v_{D_c}}$

$\bar{\sigma}_v$ = vertical effective stress

K = earth pressure coefficient

$K_{compression}$ = ~~1.0~~ 1.25 clay, 2.0 SAND

$K_{tension}$ = 0.7

table ~~4-4~~ 4-5

δ = ~~.9$\bar{\phi}$~~ 1.0 ϕ

table 4-3

A_s = side Area

For $\bar{\phi}$ = 22° ⟶ $N_{q\,Blanket}$ = 10

figure 4-4

For $\bar{\phi}$ = 30° ⟶ $N_{q\,SAND}$ = ~~25~~ 28

figure 4-4

2/

Soil Properties

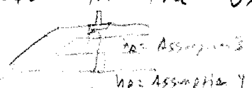
	c_u	$\bar{c}$	$\bar{\varphi}$	γ
Blanket/fill	600psf	0psf	22°	115 pcf
SAND	0psf	0psf	30°	115 pcf

Unit weights assumed same to simplify calcs

Assumptions :

- 16" × 16" square piles shown on typical sections in RD for type "N", "P" and "Q" walls are ~~not~~ NOT TAPERED PER ED-DS DRAWING ~~shown in pile details and are assumed to be tapered to 12"×12" at their tip~~

- Excess heads are computed in the underseepage analysis



- ALL calculations are for ultimate capacity of a single pile. in compression

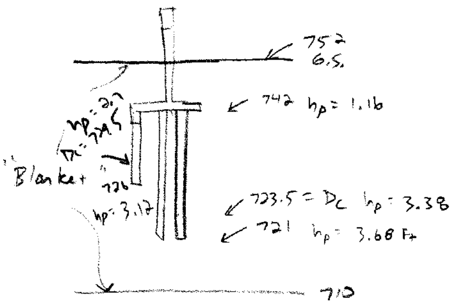
- Excess head acting at base of blankets are dissipated linearly through the blanket

- Pile and Wall details obtained from O+M Record Drawings

- Tensile capacities = 0.7 * Q_s for all piles.

3/

Station 22+81.46 to 24+54.76



Type "R" Wall
21' Long Piles
□ 18" × 18", tapered to 12" × 12"
AVG = 15" × 15"
6' o.c.

SAND HGL in SAND = 757

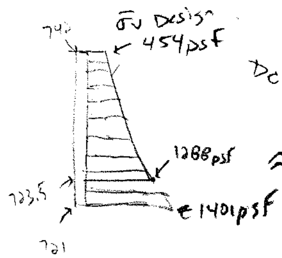
Undrained Analysis:

$Q_t = 9CA_t$
$= (9)(600 psf)(1' \times 1')$
$Q_t = 5400$ lb

$Q_s = \alpha C A_s$
$= (.9)(600 psf)(4 \times \frac{15''}{12''} \times 21')$
$Q_s = 56,700$ lb

$\boxed{Q_{ult} = 62,100 \text{ lb}}$

Drained Analysis:



$D_c = 15 \frac{15''}{12''} \times 15$
$= 18.75$ Ft
≈ el 723.5

GMB

$Q_t = \bar{\sigma}_v N_q A_t$
$= (1400 psf)(10)(1' \times 1')$
$Q_t = 14,000$ lb

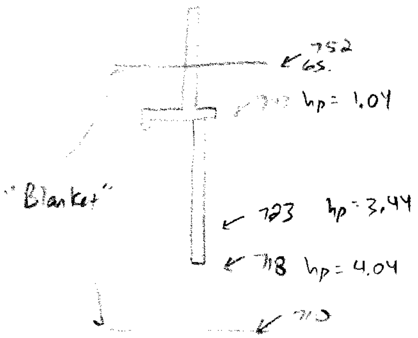
$Q_s = \bar{\sigma}_v K \tan\delta A_s$
$= (\cancel{1.0}\ 1.25)(\tan \cancel{.9 \times 22°}\ 1.0 \times 22°)\left[\left(\frac{454+1401}{2} \times 4 \times \frac{15''}{12''} \times 21' \text{Ft}\right) \cancel{+ \left(1288 \times 4 \times \frac{15}{12''} \times 2.5'\right)}\right]$
$Q_s = \cancel{35062}\ 49,183$ lb

$\boxed{Q_{ult} = \cancel{49,062}\ 63,184 \text{ lb}}$

 DRAINED : 179 UNDRAINED

4/

Station 24+54.76 to 25+38.76



Type "Q" wall, 25' long piles
□ 16" × 16" concrete pile
4' o.c.

SAND, N60 = 757

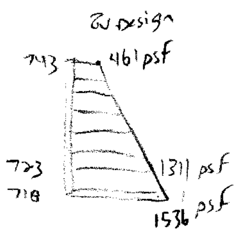
Undrained Analysis:

$Q_t = 9 c A_t$
$= (9)(600\ psf)(\frac{16}{12} \times \frac{16}{12})$
$Q_t = 9600\ lb$

$Q_s = \alpha c A_s$
$= (.9)(600\ psf)(4 \times \frac{15}{12} \times 25')$
$Q_s = 67{,}500\ lb$

$\boxed{Q_{ult} = 77{,}100\ lb}$

Drained Analysis:



~~$D_c = \frac{16}{12} \times 15 = 20' = el\ 723$~~

$Q_t = \bar{\sigma}_v N_q A_t$
$= (1536\ psf)(10)(\frac{16}{12} \times \frac{16}{12})$
$Q_t = 27{,}307\ lb$

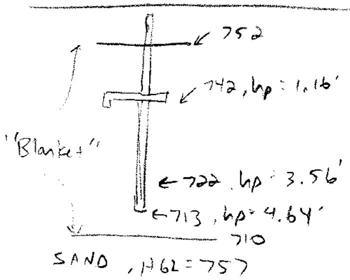
$Q_s = \bar{\sigma}_v K \tan\delta A_s$
$= (\cancel{1.0}\ 1.25)(\tan(\cancel{.94}\ 1.0 \cdot 32))(4)(\frac{16}{12}) \times [\frac{461 + \cancel{1311}\ 1536}{2} \cdot \cancel{20}\ 25 + \cancel{1311 \times 5}]$
$Q_s = \cancel{71{,}731}\ 87{,}237\ lb$

$\boxed{Q_{ult} = \cancel{75{,}238}\ 94{,}544\ lb}$

DRAINED = .98 UNDRAINED

5/

Station 25+38.76 to 26+22.76



Type "P" Wall, 29' piles
□ 16"×16" concrete piles
5' O.C.

Undrained Analysis:

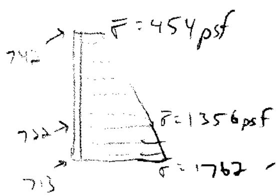
$Q_t = 9CA_t$
$= (9)(600 psf)(\frac{16}{12} \cdot \frac{16}{12})$
$Q_t = 9600$ lb

$Q_s = \alpha C A_s$
$= (0.9)(600psf)(4)(29)(\frac{16}{12})$
$Q_s = 83,520$ lb

$\boxed{Q_{ult} = 93,120 \text{ lb}}$

Drained Analysis:

$Q_t = \bar{\sigma}_v N_q A_t$
$= (\cancel{1356}\ 1762 \text{ psf})(10)(\frac{16}{12} \times \frac{16}{12})$
$Q_t = 31324$ lb



$D_c = \frac{16}{12} \times 15$
$= 20'$
@ el 722

$Q_s = \bar{\sigma}_v K \tan\delta A_s$
$= (\cancel{1.0}\ 1.75)(\tan(\cancel{24}\ 1.0 \times 22°))(4)(\frac{16}{12})\left[\left(\frac{454 + \cancel{1356}\ 1762}{2}\right) \times \cancel{20}\ 29 + \cancel{(1356 \times 9)}\right]$

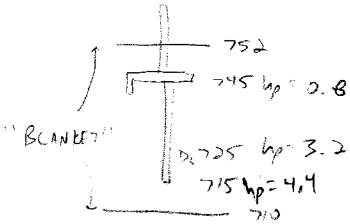
$Q_s = \cancel{61,697}\ 86,547$ lb

$\boxed{Q_{ult} = \cancel{93,021}\ 117,871 \text{ lb}}$

DRAINED = UNDRAINED

6/

Station 26+22.76 to 27+90.76



Type "N" WALL, 30' piles
□ 16" × 16" piles
6' o.c.

Undrained Analysis:

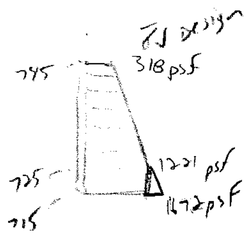
$Q_t = 9 C A_t$
$= (9)(600 psf)(\frac{16}{12} \times \frac{16}{12})$
$Q_t = 9600$ psf

$Q_s = \alpha C A_s$
$= (0.9)(600 psf)(\frac{16}{12})(4)(30)$
$Q_s = 86,400$ lb

$\boxed{Q_{ult} = 96,000 \text{ lb}}$

Drained Analysis:

$Q_t = \bar{\sigma}_v N_q A_t$
$= (\cancel{1322}\ 1672 psf)(10)(\frac{16}{12} \cdot \frac{16}{12})$
$Q_t = \cancel{21707}\ 29,724$ lb



~~$D_c = \frac{16}{12} \times 15 = 20 = el 725$~~

$Q_s = \bar{\sigma}_v K \tan\delta A_s$
$= (\cancel{1.0}\ 1.25)(\tan \cancel{24}\ 1.2 + 22°)(4)(\frac{16}{12})\left[\frac{318 + 1672}{2} \times \cancel{20}\ 30 \ \cancel{+ 1221 \times 15}\right]$
$Q_s = \cancel{57,341}\ 80,420$ lb

$\boxed{Q_{ult} = \cancel{87,040}\ \cancel{111,0124}\ 110,124 \text{ lb}}$

DRAINED = .91 UNDRAINED

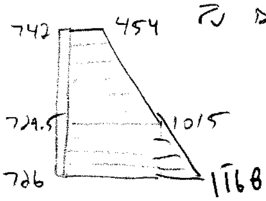
7/

Cut-off pile Station 22+81.46 to 27+90.76

DRAINED SINCE IT CONTROLS

10'' Wide Reinforced Concrete, 16' Long

$\bar{\sigma}_v$ Design



$\cancel{D_c = \frac{10''}{12} < 15 = 12.5'}$

$Q_t/ft = \bar{\sigma}_v N_q A_t$

$= (\cancel{1015}\ 1168)(10)\left(\frac{10}{12}\right)$

$Q_t/ft = \cancel{9458}\ 9733$ lb/ft

$Q_s/ft = \bar{\sigma}_v K \tan\delta A_s$

$= \left[\left(\frac{454 + \cancel{1015}\ 1168}{2}\right)(\cancel{12.4}\ 16)\right] \cancel{+ (1015 + 3.5)} *$

$\cancel{1.15}\ 1.25 * 2 * 1\ ft * \tan(\cancel{.9}\ 1.0(22))$

sides

$Q_s/ft = \cancel{9343}\ \cancel{9116}$ lb/ft

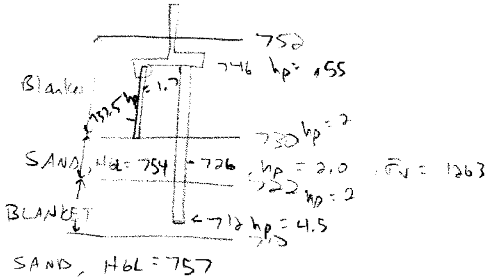
13,106

$$\boxed{Q_{tot} = \cancel{17,574}\ \cancel{19,076}\ 22,839 \text{ lb/ft}}$$

see page 3 for Drawing

8/

Station 27+90.76 to 30+42.75



$\bar{\sigma}_v = 1263$

Type "N" WALL, 34' piles
□ 16" × 16" piles
6' O.C.

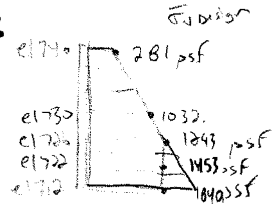
Undrained Analysis:

$Q_t = (9)(C)(A_t)$
$= (9)(600\ psf)(\frac{16}{12} \times \frac{16}{12})$
$Q_t = 9600\ lb$

$Q_s = f_s A_s = \Sigma \alpha C A_s + \Sigma \bar{\sigma}_v K \tan\delta A_s$
$= [(0.9)(600\ psf)(16') + (0.9)(600\ psf)(10') + (1263\ psf)(1.0)(\tan .9 \times 22°)(8')]\ 4 \times \frac{16}{12}$
$= [8640 + 5400 + 3638]\ 4 \times \frac{16}{12}$
$Q_s = 94,283\ lb$

$\boxed{Q_{ult} = 103,883\ lb}$

Drained Analysis:



~~$D_c = \frac{16}{12} \times 15 = 20' = el\ 726$~~

$Q_t = \bar{\sigma}_v N_q A_t$
$= (\cancel{1243}\ 1840\ psf)(10)(\frac{16}{12} \times \frac{16}{12})$
$Q_t = 32,717\ lb$

$Q_s = \Sigma \bar{\sigma}_v K \tan\delta A_s$
$= (\cancel{1.3}\ 1.0)(4)(\frac{16}{12})\left[1.25 \times \left(\frac{\cancel{1032}+281}{2} \times 16' \times \tan(\cancel{.9} \times 22°)\right) + \left(2.0 \times \frac{1453+1032}{2} \times 8 \times \tan(1.0 \times 30)\right) + \cancel{(1243 \times 8' \times \tan(.9 \times 30))}\right.$
$\left. + 1.25 \times \frac{1840+1453}{2} \times 10 \times \tan(1.0 \times 22) + \cancel{(1243 \times 10' \times \tan(.9 \times 22))}\right]$
$= \cancel{5.3(3783 + 5066 + 4142)}$
$5.3(5305 + 11,478 + 8315)$
$Q_s = \cancel{69,234}\ \cancel{78,302}\ lb$ 133,019

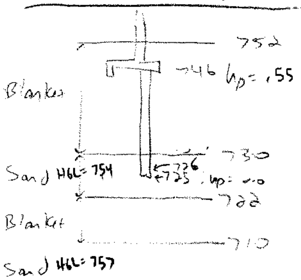
$\boxed{Q_{ult} = \cancel{101,945}\ \cancel{111,013}\ 165,730\ lb}$

DRAINED > Undrained

9/

Station 30+42.75 to 32+52.76, worst case @ 30+42.75

Type "N" Wall, 21' piles
□ 16"×16"
6' O.C.



Undrained Analysis:

use Blanket since only separated by 3×B.

$Q_t = (9)(C)(A_t)$
$= (9)(600\,psf)(\frac{16}{12}\times\frac{16}{12})$
$Q_t = 9600$ lb

$Q_s = \alpha C A_s + \bar{\sigma}_v K \tan\delta A_s$
$= (0.9)(600\,psf)(4)(\frac{16}{12})(16)$
$+ (1.0)(\tan .9\cdot 22)(4)(\frac{16}{12})\left[\frac{1032+1263}{2}\cdot 4 + 1263\cdot 1\right]$

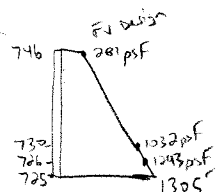
$Q_s = 46080 + 11238$
$Q_s = 57{,}318$ lb

$\boxed{Q_{ult} = 66{,}918\ lb}$

Drained Analysis:

use Blanket since only separated by 3×B

$Q_t = \bar{\sigma}_v N_q A_t$
$= (\cancel{1263}\ 1305\,psf)(10)(\frac{16}{12}\times\frac{16}{12})$
$Q_t = 23{,}200$ lb



~~Dc = 16/12 × 15 = 20' @ 726~~

$Q_s = \Sigma \bar{\sigma}_v K \tan\delta A_s$ ↗ 1.25×
$= \cancel{(1.1)}(4)(\frac{16}{12})\left[\left(\frac{281+1032}{2}\right)\tan(\cancel{.9}\cdot 22)\cdot 16\right) + \left(\frac{1305+1032}{2}\right)\tan(\cancel{.9}\cdot 32)\ 5\ \cancel{\cdot 4}$ ↗ 2.2×
$+ \cancel{\tan(.9\cdot 32)}$

$= (1)(4)(\frac{16}{12})\ \frac{5305}{\cancel{5782}} = \cancel{2301} + \cancel{2970}\ 6746$

$Q_s = \cancel{35{,}861}$ lb ~~36,017~~ lb 64,272

$\boxed{Q_{ult} = \cancel{59{,}217}\ 87{,}472\ lb}$

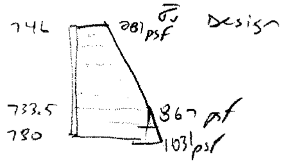
DRAINED = .88 UNDRAINED

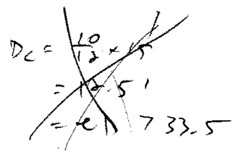
10/

Cut-off pile station 27+90.76 to 30+42.75

10" wide Reinforced concrete pile, 16' Long

Drained capacity only.





$Q_t = \bar{\sigma}_v N_q A_t$ 78

Use N_q SAND

B/L Tip at Sand and Blanket

78B Down

$= (\cancel{867}\ 1031\ psf)(\cancel{26})(\frac{10"}{12"})$

$Q_t/ft = \cancel{18,785}\ \cancel{22338}\ lb/ft$

24056

$Q_s = \bar{\sigma}_v K \tan\delta A_s$

$= \left[\left(\frac{\cancel{867}\,1031+281}{2}\right)\cancel{12.5}\ 16 + (\cancel{867)(3.5)}\right] \times \cancel{1.0}\ 1.25 \times \tan(\cancel{.8}\ 1.0 \times 27) \times 2 \text{ sides} \cdot 1ft$

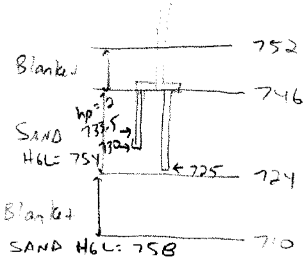
$\boxed{Q_s/ft = \cancel{2351}\ \cancel{7558}\ lb/ft}$ 10602

$$\boxed{Q_{tot} = \cancel{26,136}\ \cancel{29,096}\ 34,658\ lb/ft}$$

See page 8 for Drawing

11/

Station 32+52.76 to 48+06.76



Type "N" Wall, 21' piles
□ 16" × 16" 6' O.C
32+52.76 → 42+18.76

Type "P" wall, 21' piles
□ 16" × 16" 5' O.C
42+18.76 → 43+86.76

Type "Q" wall, 21' pile
□ 16" × 16" 4' O.C
43+86.76 → 48+06.76

Undrained Analysis:

Use Blanket due to 1' separation

$Q_t = 9 C A_t$

$= (9)(600\,psf)\left(\frac{16}{12} \times \frac{16}{12}\right)$

$Q_t = 9600$ psf

$\boxed{Q_{ult} = 5{,}941 \text{ lb}}$

$Q_s = \bar{\sigma}_v K \tan\delta A_s$

$= \left(\frac{190.8 + 1243}{2}\right)(20) + (1243 \times) \times \frac{2.0}{20} \times$

$[\tan(\cancel{28}\,30)] \times 4 \times \frac{16}{12}$

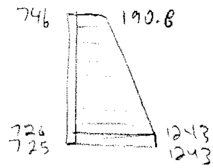
$Q_s = \cancel{42,311}$ 95,954

$D_c = \frac{16}{12} \times 15$

$= 20'$

$=$ el 726

$\bar{\sigma}_v$ design



DRAINED Analysis:

$Q_t = \bar{\sigma}_v N_q A_t$

$= (1243)(10)\left(\frac{16}{12} \times \frac{16}{12}\right)$

$Q_t = 22097$ lb

$Q_s = Q_s$ from above

$Q_s = \cancel{42,354}$ 95,954

$\boxed{Q_{ult} : \cancel{64,459}\ 118{,}051}$

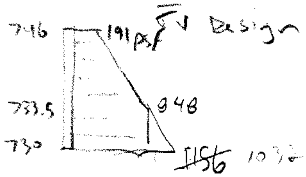
$D_c \approx$ end of pile, No Revision

12/

Cut-off pile   station 32+52.76 to 48+06.76

10" wide concrete pile 16' Long

IN SAND SO DRAINED = UNDRAINED



USC SAND END BEARING

CAUSE Blanket ≈ 8B AWAY

$Q_t/ft = \bar{\sigma}_v N_q A_t$

$= (\cancel{848}\ \cancel{1156}\ 1032\ psf)(\cancel{26}\ 28)\left(\frac{10''}{12''}\right)$

$Q_t/ft = \cancel{18{,}373}\ 25{,}054$ lb/ft

~~21,500~~

24,080

$Q_s/ft = \bar{\sigma}_v K \tan\delta A_s$

$= \left[\frac{191 + \cancel{848}\ \cancel{1156}\ 1032}{2}(\cancel{27}\ 16) + \cancel{(848)(3.5)}\right] + \cancel{1.8}\ 2.0 + (\tan \cancel{\phi}\ 1.0\ (30)) + 1'$ 2 sides

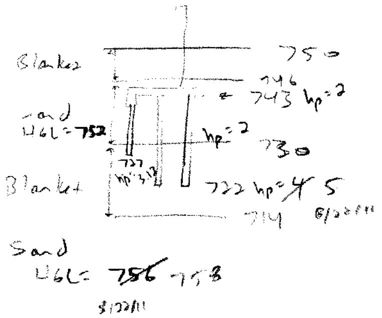
$Q_s/ft = \cancel{9648}\ 10981$ lb/ft

22,595

$$\boxed{Q_{total} = \cancel{28{,}015}\ \cancel{36{,}035}\ \cancel{32{,}322}\ 46{,}675 \text{ lb/ft}}$$

13/

Station 48+06.76 to 60+24.76 — Type "S" wall, 21' piles ☐ 18"x18", tapered 4' o.c. 2 Rows

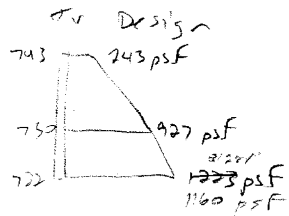


Type "R" Wall, 21' piles

☐ 18"x18", tapered to 12"x12"

6' O.C., 2 Rows   Avg= 15"x15"

$\bar{\sigma}_v$ Design



$D_c = \frac{15}{12} \times 15$

$= 19$

$D_c \approx$ Pile length.

Undrained Analysis

$Q_t = 9 c A_t$

$= (9 \cdot 1600 \text{ psf})(\frac{12}{12} \cdot \frac{12}{12})$

$Q_t = 5400$ lb

$\boxed{Q_{tot} = \cancel{40,255}\ 46,660 \text{ lb}}$

$Q_s = \bar{\sigma}_v K \tan\delta A_s$

$= \left[\left[\left(\frac{927+243}{2}\right)(13) \tan(\cancel{1.0}\,2.0 \cdot 30°)\right] + \left[\left(\frac{\cancel{1223}\,1160+927}{2}\right)(6) \tan(\cancel{1.0}\,1.25 \cdot 22°)\right]\right]$

$+ (\cancel{x}) \times 4 \cdot \frac{15}{12}$

$= (\cancel{3875}\ 8761 + \cancel{3046}\ 4216)\ 5$

$Q_s = \cancel{34,855}$ lb

~~34,100~~ lb

64,765

Drained Analysis

$Q_t = \bar{\sigma}_v N_q A_t$

$= (\cancel{1223}\ 1160 \text{ psf})(10)(\frac{12}{12} \cdot \frac{12}{12})$

$Q_t = \cancel{12,230}\ 11,600$ lb

$Q_s =$ Same as above

$\boxed{Q_{tot} = \cancel{47,085}\ 76,565 \text{ lb}}$ ~~46,000~~

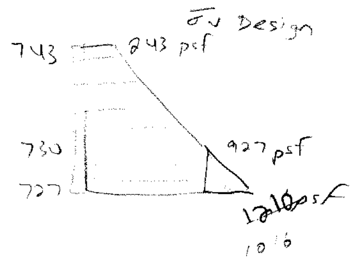
$D_c \approx$ pile length

No Revision.

14/

Cut-off pile $48+06.76$ to $60+24.76$

10" wide concrete pile, 16' long



Undrained Analysis

$Q_t/ft = 9 C A_t$
$= (9)(600 psf)(\frac{10}{12}/ft)$

$Q_t/ft = 4508$ lb/ft

$Q_s = f_s A_s = \underset{\text{clay}}{\alpha C A_s} + \underset{\text{SAND}}{\bar{\sigma}_v K \tan\delta A_s}$

$Q_s/ft = (.9)(600 psf)(2 sides)(3') + \left(\frac{927+243}{2}\right)(\cancel{1.0}\ 2.2)(\tan \cancel{A}\ 1.0 \times 32)(2 sides)(13')$

$Q_s/ft = 3240 + \cancel{7750}\ 17,563$

$Q_s/ft = 10,990$ lb/ft

$\boxed{Q_{tot} = 15,498 \text{ lb/ft}}$

Drained Analysis

$Q_t = \bar{\sigma}_v N_q A_t$
$= (\cancel{927}\ \cancel{1210}\ 1016 psf)(10)(\frac{10}{12})$

$Q_t/ft = \cancel{7,725}\ \cancel{10,083}$ lb/ft
8,467

Same as above

$Q_s = \Sigma \bar{\sigma}_v K \tan\delta A_s$ = CLAY DRAINED + SAND DRAINED
$= \left(\frac{\cancel{1210}\ 1016+927}{2}\right)(\cancel{1.25}\ 1.0)(\tan \cancel{A}\ 1.0 \times 22)(3.)\times 2 sides + \cancel{7750}\ 17,563$

$= \cancel{2308}\ \cancel{2008}\ 7939 + \cancel{7750}\ 17,563$

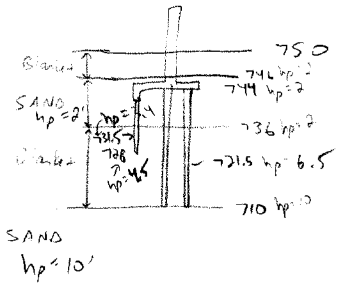
$Q_s/ft = \cancel{10058}\ \cancel{9,814}$ lb/ft 20,502

$\boxed{Q_{tot} = \cancel{17,477}\ 20141 \text{ lb/ft}}$

~~16,176~~ 5-74, 28,969

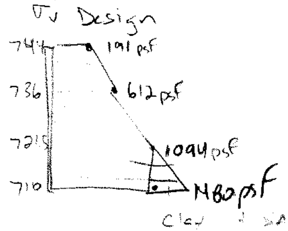
15/

Station 60+24.76 to 73+2014



Type "R" Wall
Avg Piles = 34'
PJ 18"x18" straight
6' o.c. 2 Rows

$D_c = \frac{18}{12} \times 15$
$= 22.5$
$= 721.5$



Undrained Analysis:

Use tip in Blanket B/C clay.

$Q_t = 9CA_t$
$= (9)(600\text{psf})\left(\frac{18\times18}{144}\right)$
$Q_t = 12150 \text{ lb}$

$\boxed{Q_{tot} = 106{,}210 \text{ lb}}$

Clay + SAND
$Q_s = \alpha C A_s + \bar{\sigma}_v K \tan\delta A_s$
$= (.9)(600\text{psf})(4)\left(\frac{18}{12}\right)(26) +$
$\left(\frac{612+191}{2}\right)(2.0)(\tan^{1.0} 22)(4)\left(\frac{18}{12}\right)(8)$
$Q_s = 84{,}240 + \cancel{9820}\ 22254$
$Q_s = 94060 \text{ lb}$

Drained Analysis:

$Q_t = \bar{\sigma}_v N_q A_t$
$= (\cancel{1094}\ 1480\text{psf})(10)\left(\frac{18\times18}{144}\right)$
$Q_t = \cancel{24615 \text{ lb}}\ 33{,}300 \text{ lb}$
$Q_s = \Sigma \bar{\sigma}_v K \tan\delta A_s$ (split out clay and sand)

sand same as above

$Q_s = \cancel{9820}\ 22{,}254 \text{ lb} + \left[\left(\frac{\cancel{1094}\ 1480+612}{2}\right)(\cancel{14.5}\ 26) + \cancel{(1094)(11.5)}\right](\cancel{1.0}\ 1.25)(\tan^{1.0} 22)(4)\left(\frac{18}{12}\right)$

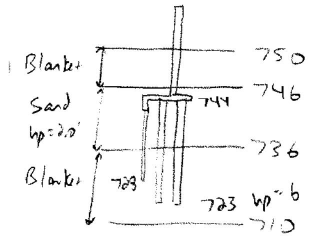
$= \cancel{9820}\ 22254 + \cancel{53894}\ \cancel{53340}\ 82409$

$Q_s = \cancel{63{,}714}\ \cancel{60{,}567} \text{ lb}\ 104{,}663$

$\boxed{Q_{total} = \cancel{83{,}329}\ \cancel{101{,}867}\ 137{,}963 \text{ lb}}$

(b)

Station 73+20.14 to 78+12.22



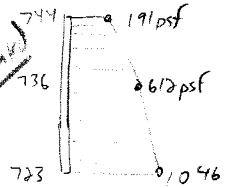
Type "R" Wall, 21' Piles
□ 18"×18" TAPERED
6' o.c.

Type "Q" WALL, 21' Piles
□ 16"×16" Square
4' o.c.

SAND
$\gamma_p = 10'$

$\bar{\sigma}_v$ Design

use Type "Q" WALL
in CALCS.



Undrained Analysis (not updated)

$D_c = \frac{16}{12} \cdot 15$
$= 20'$
$\approx$ Pile length

$Q_t = 9 c A_t$
$= (9)(600psf)(\frac{16}{12} \cdot \frac{16}{12})$

$Q_t = 9600\ lb$

$Q_s = \underset{SAND}{\bar{\sigma}_v K \tan\delta A_s} + \underset{CLAY}{\alpha C A_s}$
$= (\frac{612+191}{2})(\overset{2.9}{\cancel{2}})(\tan\overset{1.2}{\cancel{1}}\times 30)(4)(\frac{16}{12})(8') +$
$(.9)(600psf)(4)(\frac{16}{12})(13')$
$= \overset{19,781}{\cancel{8729}}\ lb + 37440\ lb$

$\boxed{Q_{tot} = 55,769\ lb}$

$Q_s = 46169\ lb$

Drained Analysis

$Q_t = \bar{\sigma}_v N_q A_t$
$= (1046psf)(10)(\frac{16}{12} \cdot \frac{16}{12})$

$Q_t = 18,596\ lb$

$Q_s = \Sigma \bar{\sigma}_v K \tan\delta A_s$ (split clay/sand, sand as above)
$= \overset{19,781}{\cancel{8729}} + (\frac{612+1046}{2})(\overset{1.25}{\cancel{1}})(\tan\overset{1.2}{\cancel{1}}\times 22)(4)(\frac{16}{12})(13')$
$= \overset{19,781}{\cancel{8729}} + \overset{28,028}{\cancel{20693}}$

$Q_s = \cancel{29422}\ lb$
48,809

$\boxed{Q_{total} = \overset{67,425}{\cancel{48,018\ lb}}}$

$D_c \approx$ Pile length,
No Revision.

171

Cut-off wall   Station 60+24.76 → 78+12.22

10' Wide concrete pile, 16' LONG

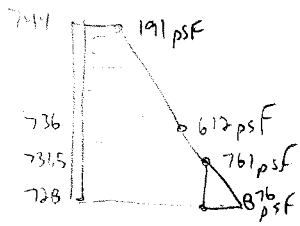
$\bar{\sigma}_v$ Design

$D_c = \frac{10}{12} \times 15$

$= 12.5$

el $= 731.5$

see page 15 for diagram



Drained Analysis since it controls other piles

$Q_t = \bar{\sigma}_v N_q A_t$

$= (\cancel{761}\ 876\ psf)(10)(10''/12) \times 1 ft$

$Q_t/ft = \cancel{6342}\ 7300$ lb/ft

$Q_s = \Sigma \bar{\sigma}_v K \tan\delta A_s$ $\times 20$ $\times 1.25$

$= \left[\left(\frac{191+612}{2}\right)(\tan \cancel{0.8}\times 30)(8) + \left[\left(\frac{612+\cancel{761}\ 876}{2}\right)(\cancel{4.5}\ 8) + \cancel{(761+\ 3.5)}\right] \tan(\cancel{A}\ 1.0 \times 22)\right] \times 2 \text{ sides}$

$= (\cancel{1037}\ 3709 + \cancel{1385}\ \cancel{2443}\ 3006) \times 2 \text{ sides}$

2103

$Q_s/ft = \cancel{6043}\ 7560$ lb/ft

13,430

$$\boxed{Q_{total}/ft = \cancel{12385}\ \cancel{14,060} \text{ lb/ft}}$$

20,730

8/08/11 Designer: G. [illegible]

CID-MO Pile Capacity For ED-05 [illegible]

18

ED-05 needs pile capacities for [illegible] -1, -2, -3 for the reliability analysis for the following sections:

Sta 48+06.76 - 60+24.76, 13" tapered pile, 21' long

Sta 48+06.76 - 60+64.76 10' cut off pile, 16' long

Sta 60+64.76 - 73+20.14 16" step-taper pile, 34' long

Sta 73+20.14 - 78+16.22 16" step-taper pile, 21' long

Sta 60+64.76 - 78+16.22 10' cut off pile 16' long

Assumptions:

- Parameters and assumptions the same as TOL pile calculations
- Only drained analysis can be performed

$Q_s = \sum \gamma K L \sigma \delta A_s$, $K = 1.0$ compression, $K = 0.7$ tension, $\delta = 0.75$

$Q_t = \sum \sigma_v N_q A_t$ $N_q = 10$ silt, $N_q = 25$ sand

Note: conservatism Removed in Revised calcs. Reliability Not Revised b/c not used.

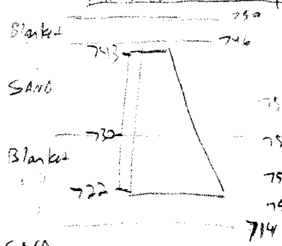
After doing 1st section, there is very little increase in capacity as the water level decreases.

Do Not increase capacities for reliability. Note as being conservative

19

Station 48+06.76 to 60+24.76   21' long piles

continued from page 18



Water elevation	h_{p743}	h_{p730}	h_{p722}	$\bar{\sigma}_{743}$	$\bar{\sigma}_{730}$	$\bar{\sigma}_{722}$
759.6 TOL	2	2	[illegible]	243	927	1160
758.6 TOL-1	2	2	4.7	243	927	1180
757.8 TOL-2	2	2	4.3	243	927	1204
756.8 TOL-3	2	2	3.8	243	927	1236

use middle sand held @ $h_p = 2'$

USE LSTAGE ELEV
TOL : 758
-1 : 757.3
-2 : 756.5
-3 : 755.6

$\underline{Tip}$: $Q_t = \bar{\sigma} N_q A_t$

$= (\bar{\sigma}_{722})(10)(\frac{12}{12})(\frac{12}{12})$

$= \bar{\sigma}_{722} \times 10$

$\underline{Side}$: $Q_s = \Sigma\, \bar{\sigma}_v K \tan\delta\, A_s$

$= \left[\frac{243+927}{2}(13)\tan(.9\times 30°) + \frac{\bar{\sigma}_{722}+927}{2}(8)\tan(.9\times 30°)\right]$

$\times 1.0 \times 4.0 \times \frac{15}{12}$

$= 19375 + (\bar{\sigma}_{722} + 927) 7.2$

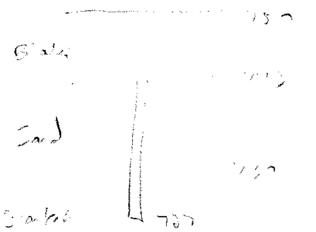
Water	Tip (lb)	Side (lb)	Total Comp (lb)	Tension (lb)
TOL	11,600	34,401	46,001	24081
TOL-1	11,800	34,545	46,345	24182
TOL-2	12,040	34,718	46,758	24,303
TOL-3	12,360	34,949	47,309	24,464

20/

Station 48+56 to 60+00

10" concrete cut off pile

continued from page 19



water	h_{p743}	h_{p730}	h_{p727}	$\bar{\sigma}_{743}$	$\bar{\sigma}_{730}$	$\bar{\sigma}_{727}$
TOL	2	2	3.1	743	987	1016
TOL-1	2	2	3.0			1023
TOL-2	2	2	2.8			1035
TOL-3	2	2	2.7			1041

Tip: $Q_t = \bar{\sigma} N_t A_t$

$= (\bar{\sigma}_{727})(10)(\frac{10}{12})$

$= \bar{\sigma}_{727} \cdot 8.3$ lb/ft

Side: $Q_s = \sum \bar{\sigma}_i \, k \tan\delta \, A$

$= \left(\frac{\bar{\sigma}_{730} + 987}{2}\right)$ [illegible] $= \left(\frac{\ldots}{2}\right)$ [illegible]

$= \bar{\sigma}_{730}$ [illegible] $1.08 + 7750$

Water	Tip lb/ft	Side lb/ft	Load Carry lb/ft	Tension lb/ft
TOL	8467	9849	18,316	6,894
TOL-1	8525	9856	18,381	6899
TOL-2	8625	9869	18,494	6908
TOL-3	8667	9875	18,542	6913

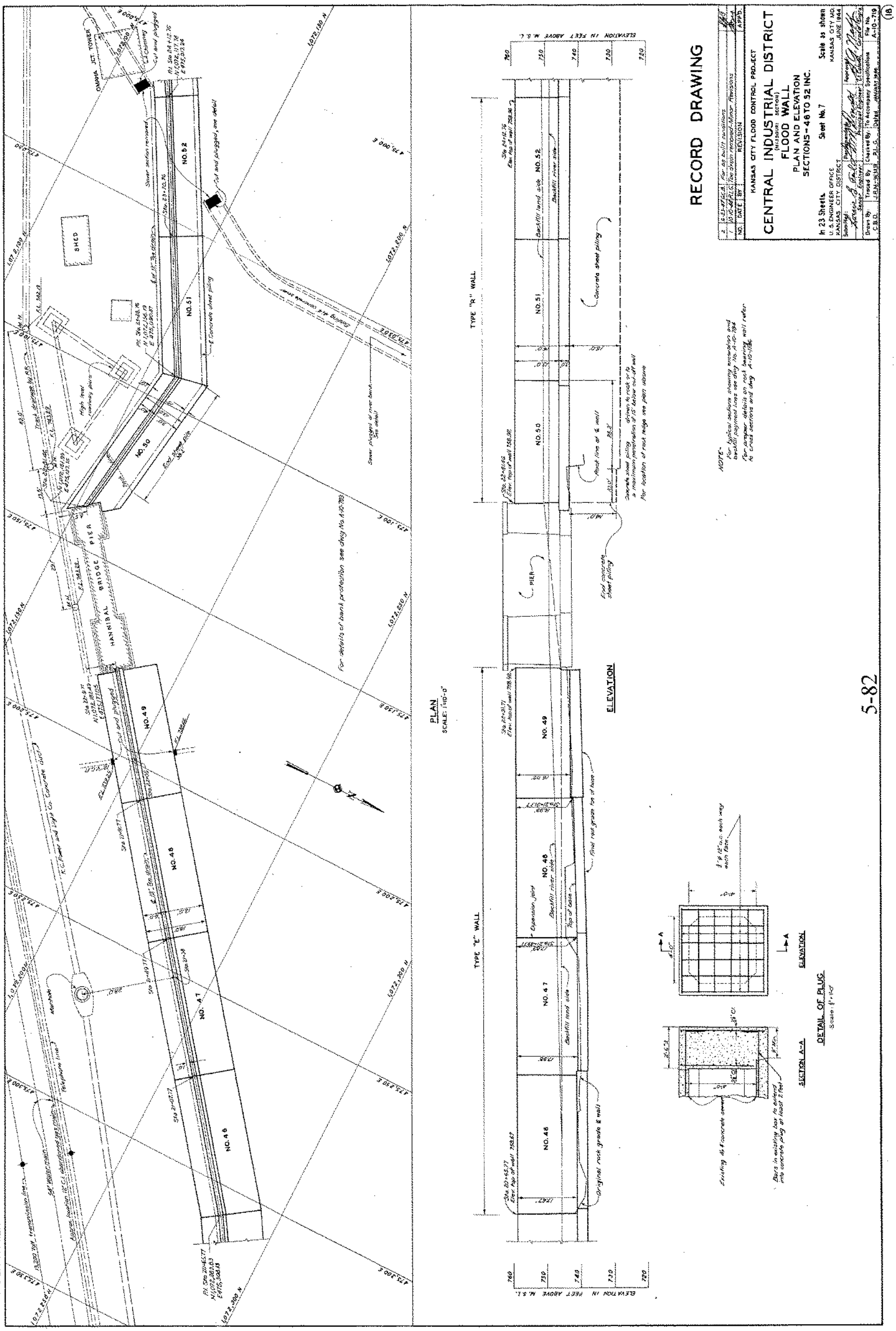
Record Drawings Used

Floodwall Plan

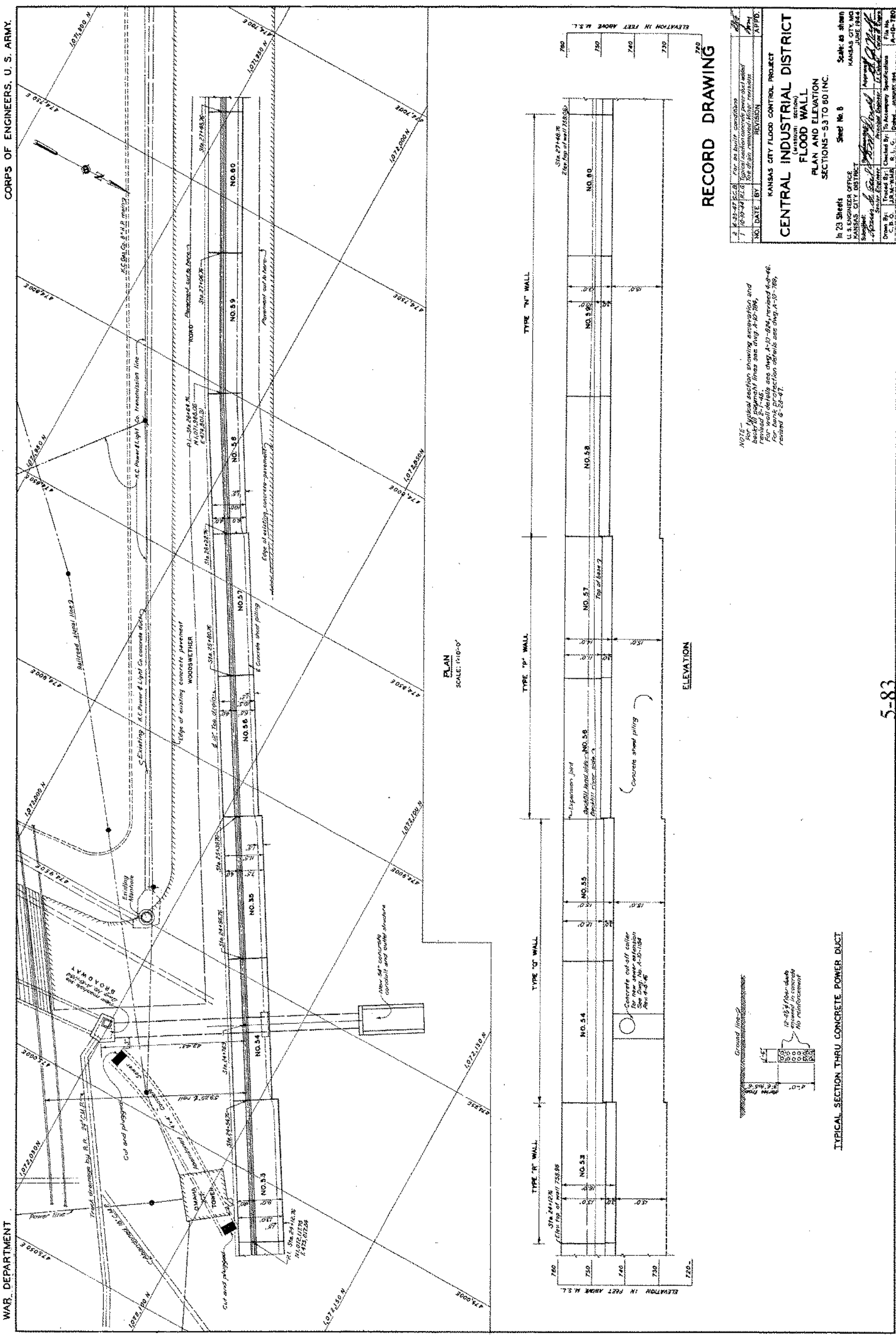
Pile Driving Records

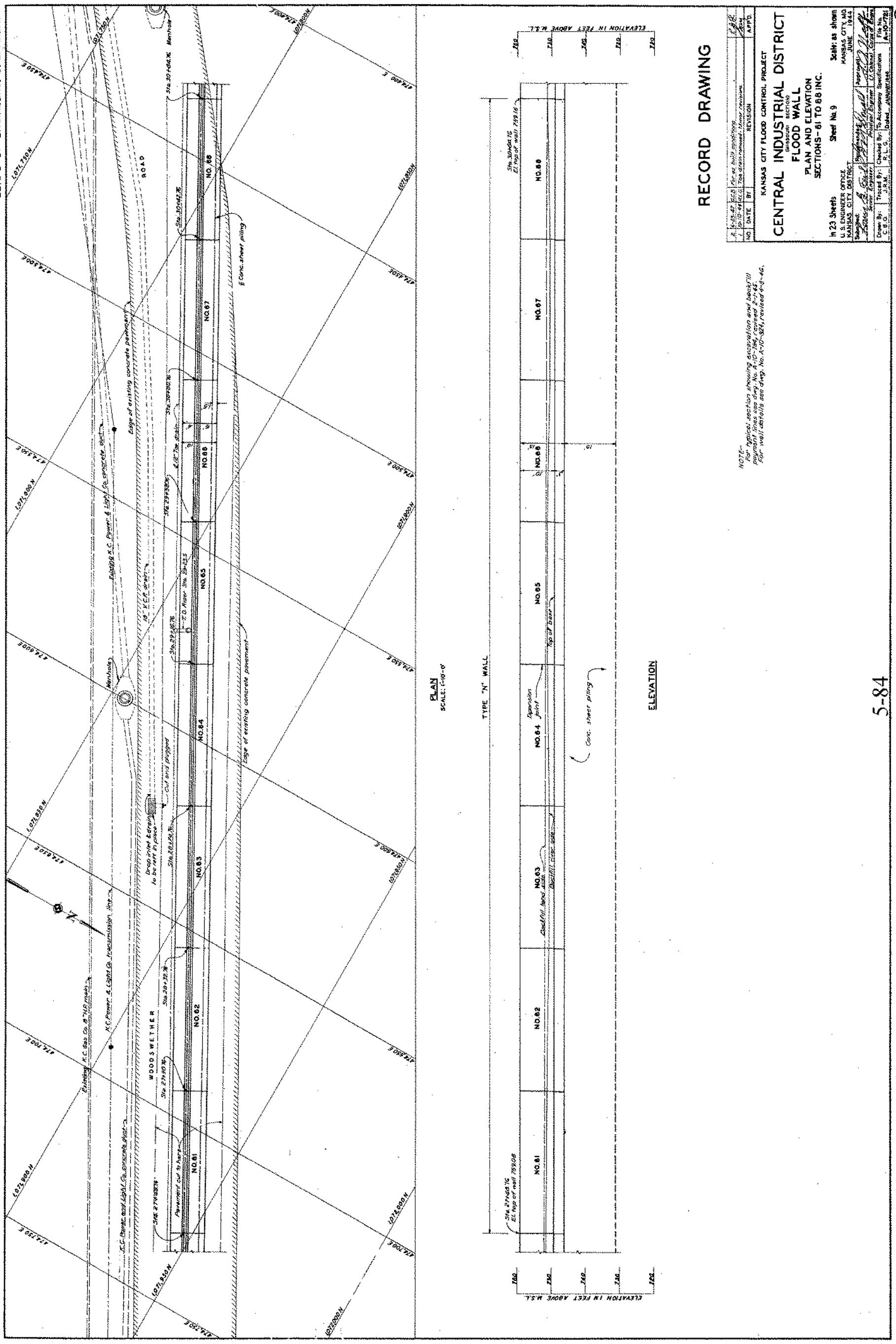
Floodwall Typical Sections

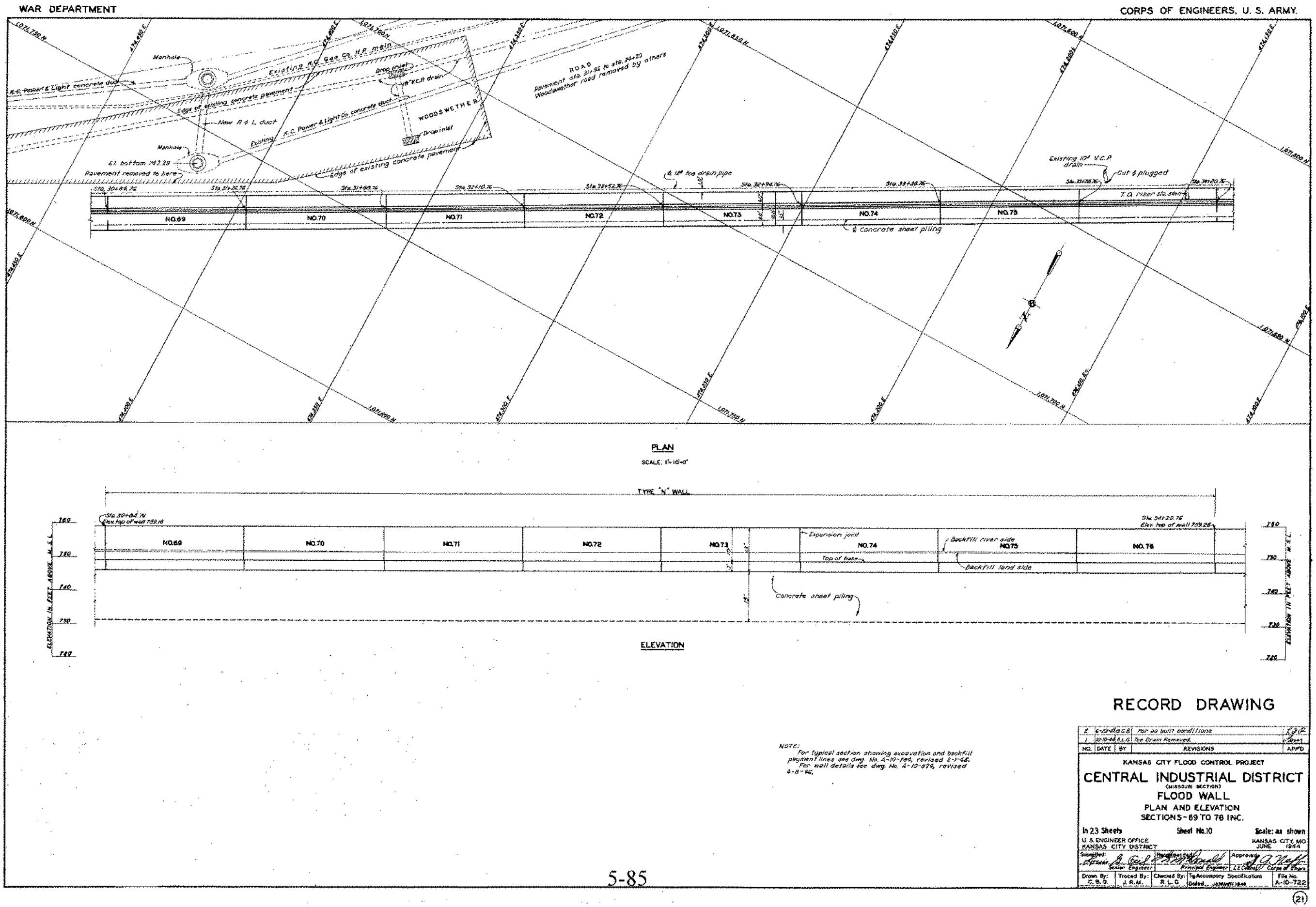
Pile Details



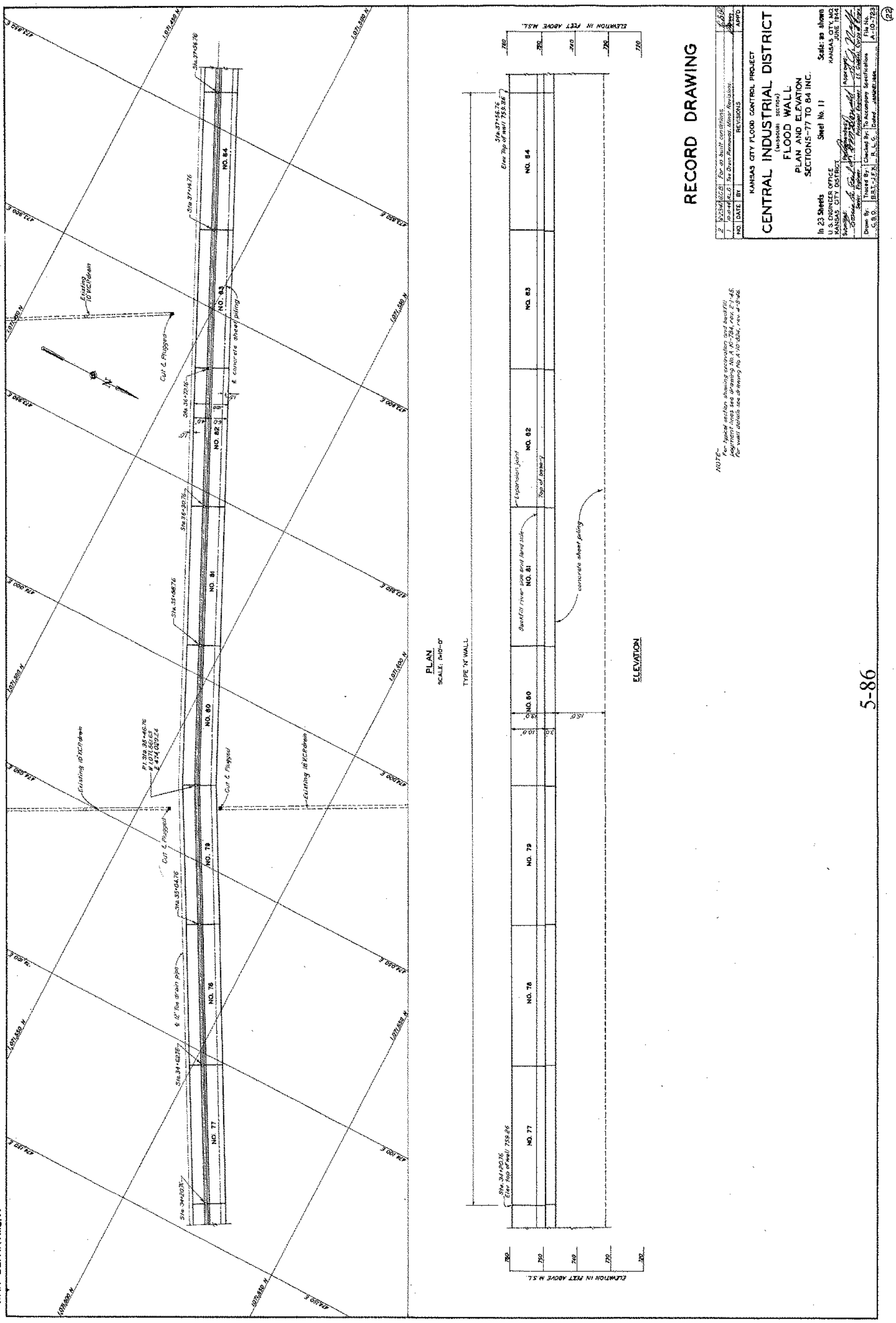
WAR DEPARTMENT
CORPS OF ENGINEERS U. S. ARMY
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-48 TO 52 INC.
PLAN
ELEVATION
TYPE "E" WALL
TYPE "R" WALL
NO. 46
NO. 47
NO. 48
NO. 49
NO. 50
NO. 51
NO. 52
PIER
HANNIBAL BRIDGE
SHED
DETAIL OF PLUG
SECTION A-A
5-82

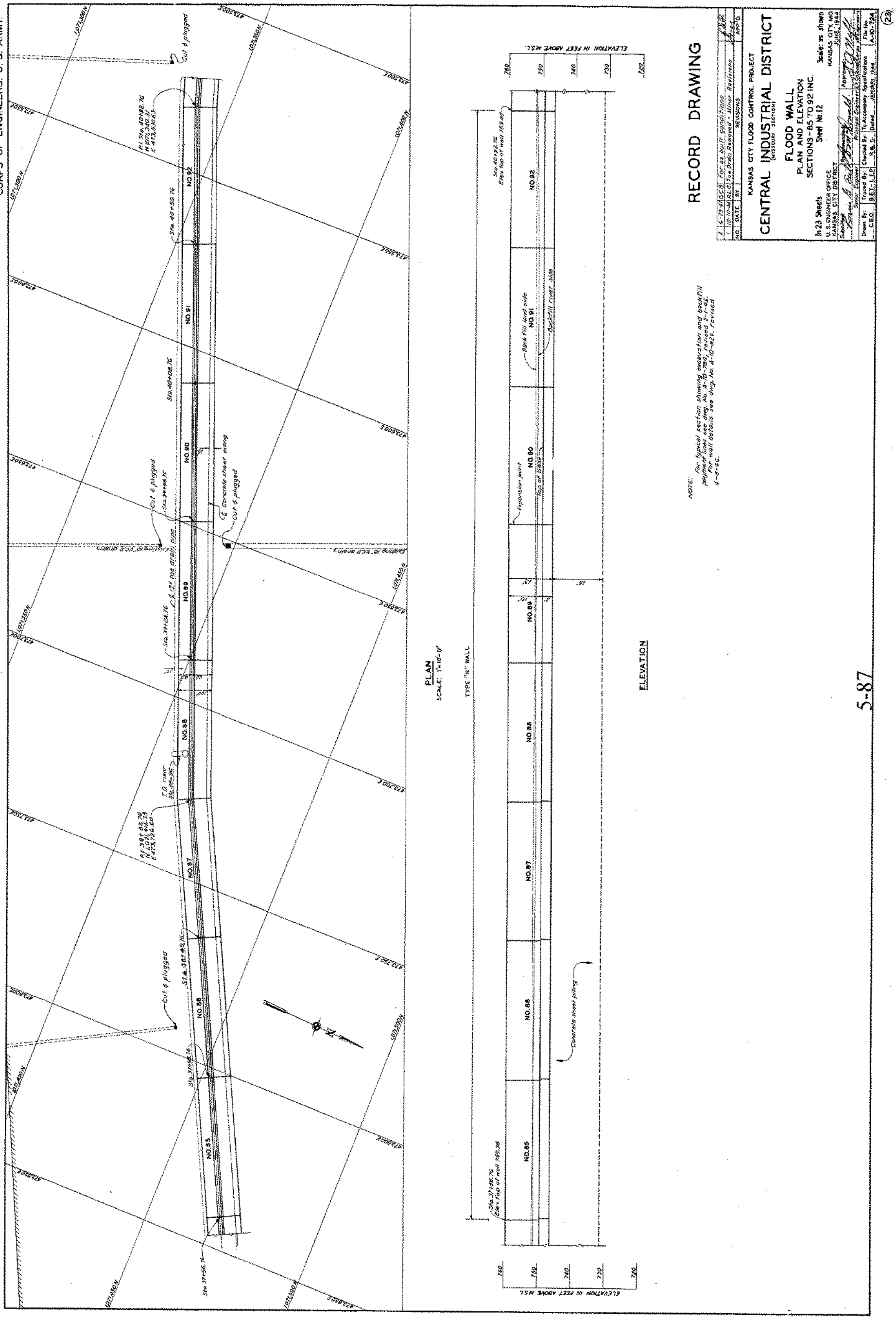

WAR DEPARTMENT
CORPS OF ENGINEERS, U. S. ARMY.
PLAN
ELEVATION
TYPE "N" WALL
TYPE "P" WALL
TYPE "Q" WALL
TYPE "R" WALL
TYPICAL SECTION THRU CONCRETE POWER DUCT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-53 TO 60 INC.
5-83

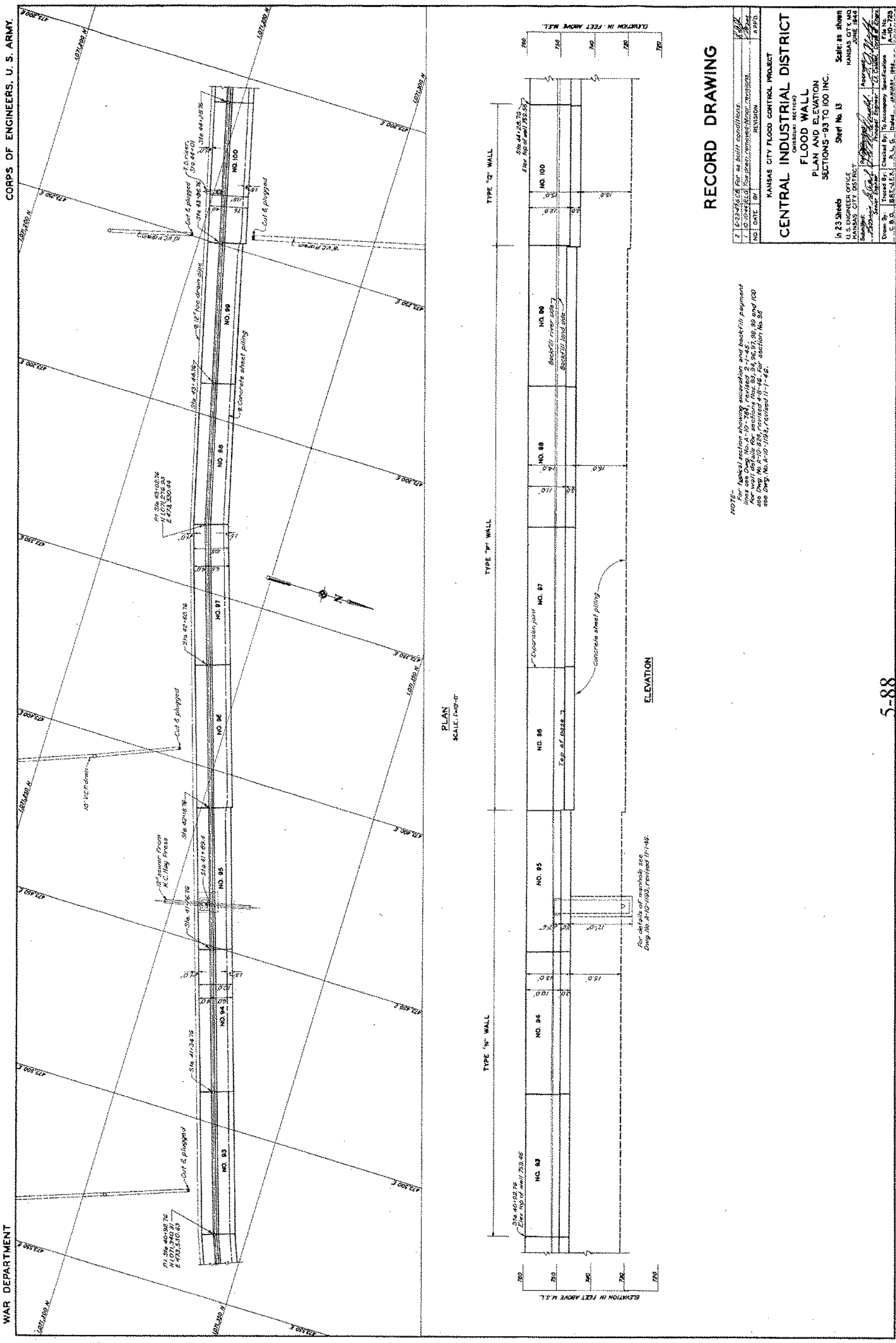

CORPS OF ENGINEERS, U. S. ARMY.
WAR DEPARTMENT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS - 61 TO 68 INC.
In 23 Sheets
Sheet No. 9
U.S. ENGINEER OFFICE KANSAS CITY DISTRICT
Scale: as shown
KANSAS CITY, MO.
JUNE 1944
PLAN
TYPE "N" WALL
ELEVATION
WOODSWETHER
ROAD
NO. 61
NO. 62
NO. 63
NO. 64
NO. 65
NO. 66
NO. 67
NO. 68
Manhole
Expansion joint
Conc. sheet piling
Top of base
ELEVATION IN FEET ABOVE M.S.L.


WAR DEPARTMENT
CORPS OF ENGINEERS, U. S. ARMY.
Manhole
K.C. Power & Light concrete duct
Existing K.C. Gas Co. H.P. main
Edge of existing concrete pavement
New P & L duct
Existing K.C. Power & Light Co. concrete duct
Drop inlet
18" K.C.R. drain
WOODSWETHER
ROAD
Pavement sta. 31+85 to sta. 34+20
Woodswether road removed by others
F.L. bottom 742.29
Pavement removed to here
Edge of existing concrete pavement
12" toe drain pipe
Existing 10" V.C.P. drain
Cut & plugged
T.O. riser
NO.69
NO.70
NO.71
NO.72
NO.73
NO.74
NO.75
NO.76
Concrete sheet piling
PLAN
SCALE: 1"=10'-0"
TYPE "N" WALL
Sta. 30+84.76
Elev. top of wall 759.16
Sta. 34+20.76
Elev. top of wall 759.25
Expansion joint
Backfill river side
Backfill land side
Top of base
Concrete sheet piling
ELEVATION IN FEET ABOVE M.S.L.
ELEVATION
NOTE:
For typical section showing excavation and backfill payment lines see dwg. No. A-10-789, revised 2-1-45.
For wall details see dwg. No. A-10-675, revised 4-8-46.
RECORD DRAWING
For as built conditions
Toe Drain Removed.
NO. DATE BY REVISIONS APP'D
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
(MISSOURI SECTION)
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-69 TO 76 INC.
In 23 Sheets
Sheet No.10
Scale: as shown
U. S. ENGINEER OFFICE
KANSAS CITY DISTRICT
KANSAS CITY, MO.
JUNE 1944
Submitted:
Senior Engineer
Recommended:
Principal Engineer
Approved:
Corps of Engrs.
Drawn By: C.B.O.
Traced By: J.R.M.
Checked By: R.L.G.
To Accompany Specifications
File No. A-10-722
(21)

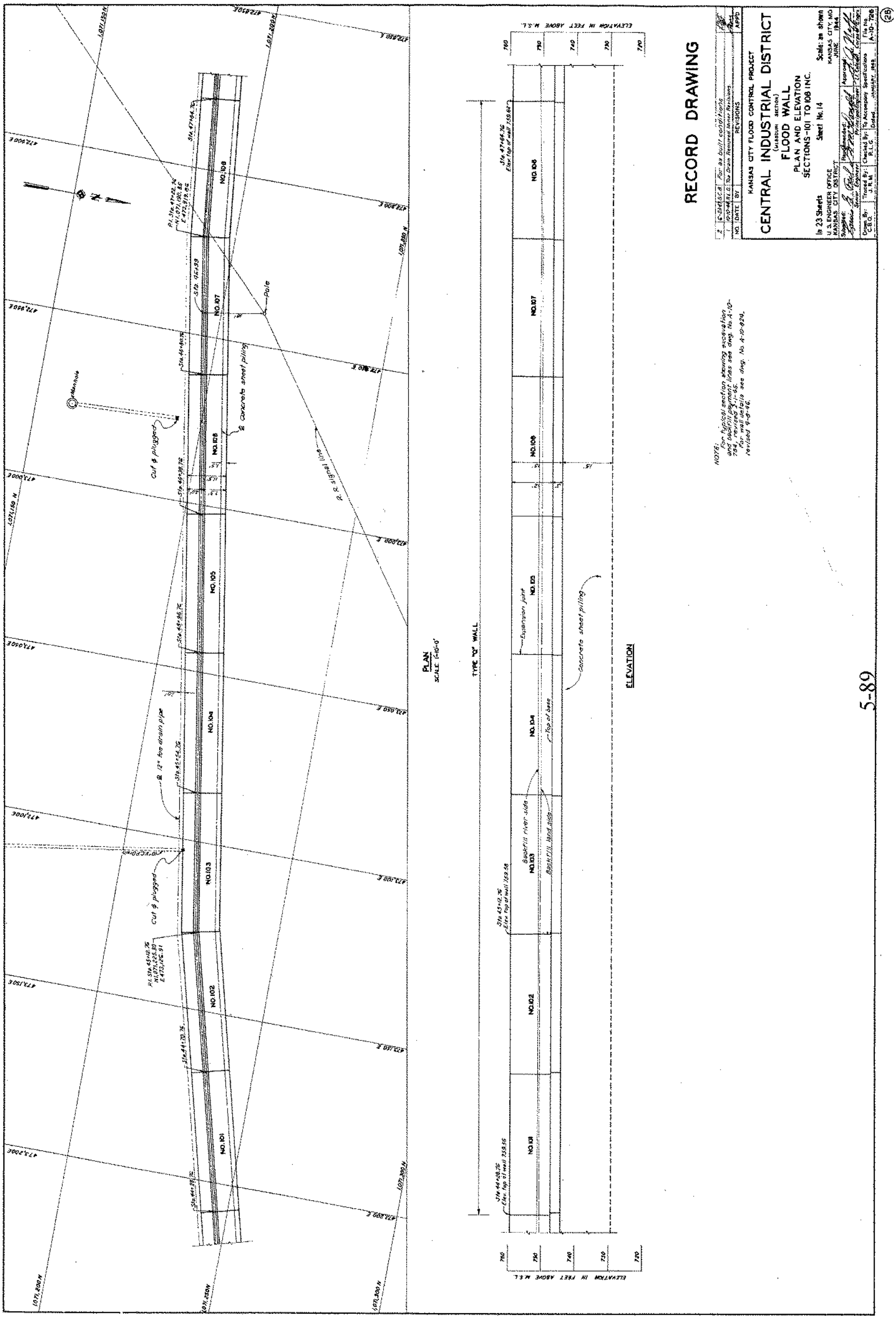
5-85

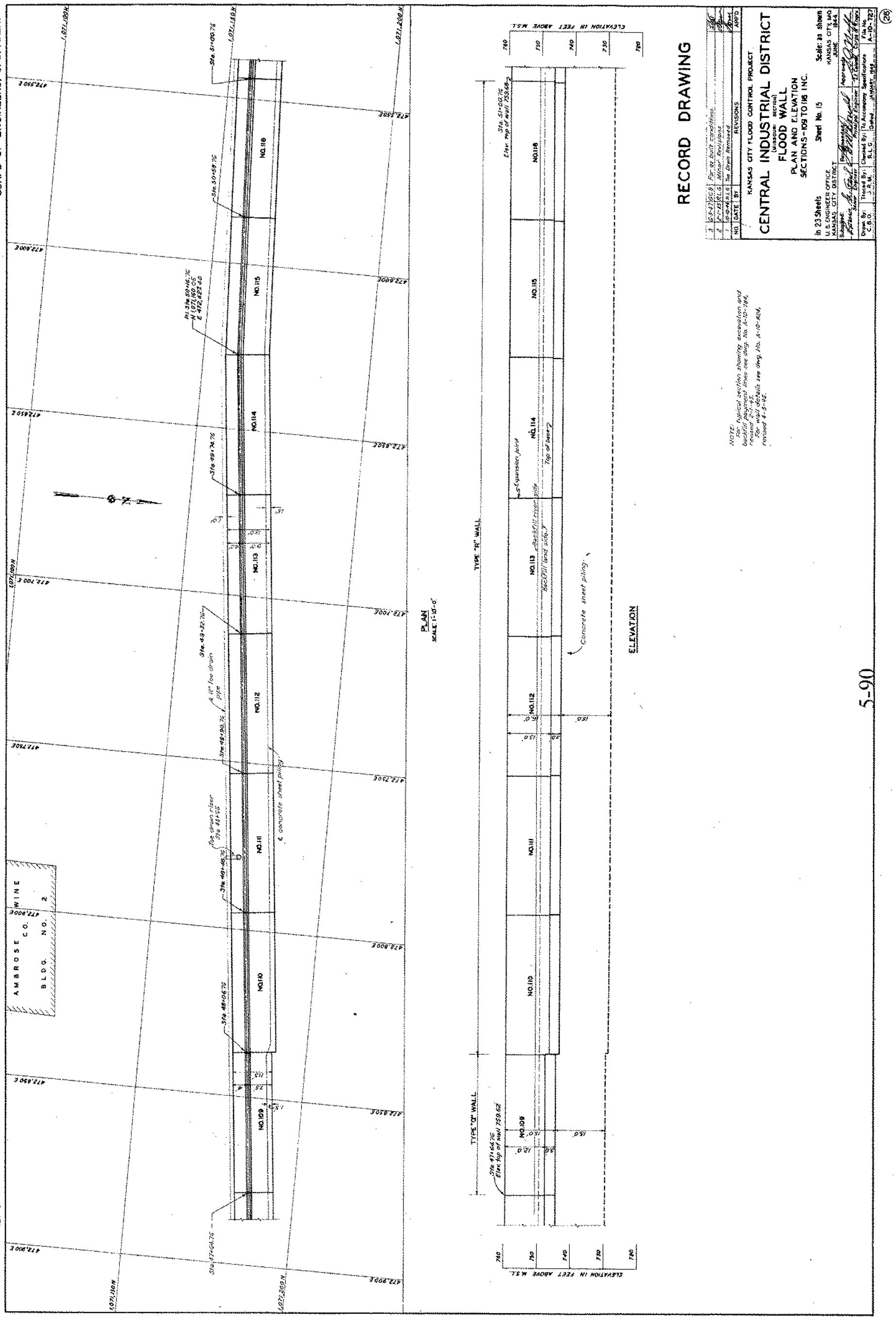

WAR DEPARTMENT
CORPS OF ENGINEERS, U. S. ARMY.
PLAN
TYPE "N" WALL
ELEVATION
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-77 TO 84 INC.
In 23 Sheets
Sheet No. 11
U. S. ENGINEER OFFICE
KANSAS CITY DISTRICT
5-86

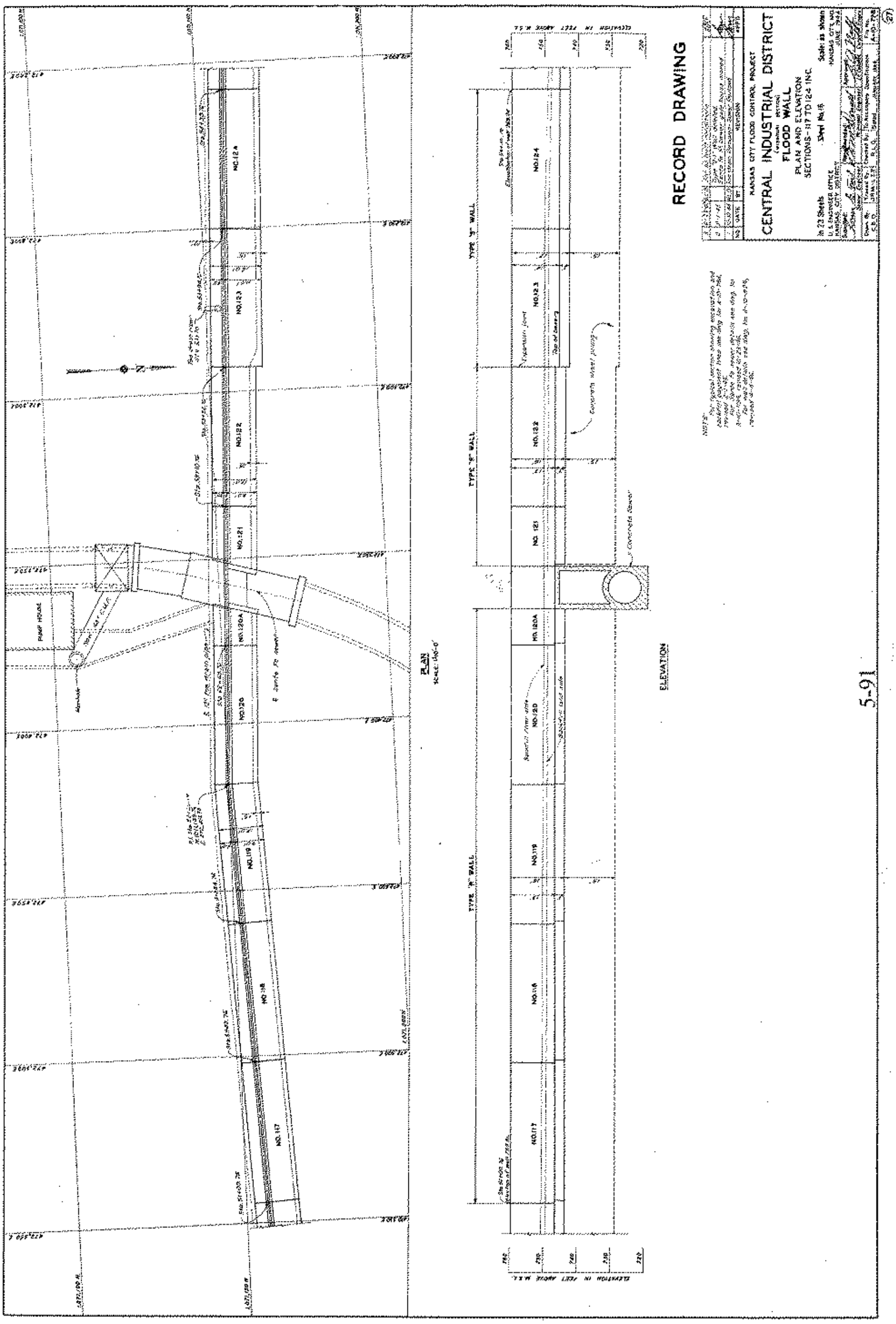

WAR DEPARTMENT
CORPS OF ENGINEERS. U. S. ARMY.
PLAN
SCALE: 1"=10'-0"
TYPE "N" WALL
ELEVATION
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
(MISSOURI SECTION)
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-85 TO 92 INC.
Sheet No.12
In 23 Sheets
Scale: as shown
U.S. ENGINEER OFFICE
KANSAS CITY DISTRICT
KANSAS CITY, MO.
JUNE, 1944
A-10-724
5-87

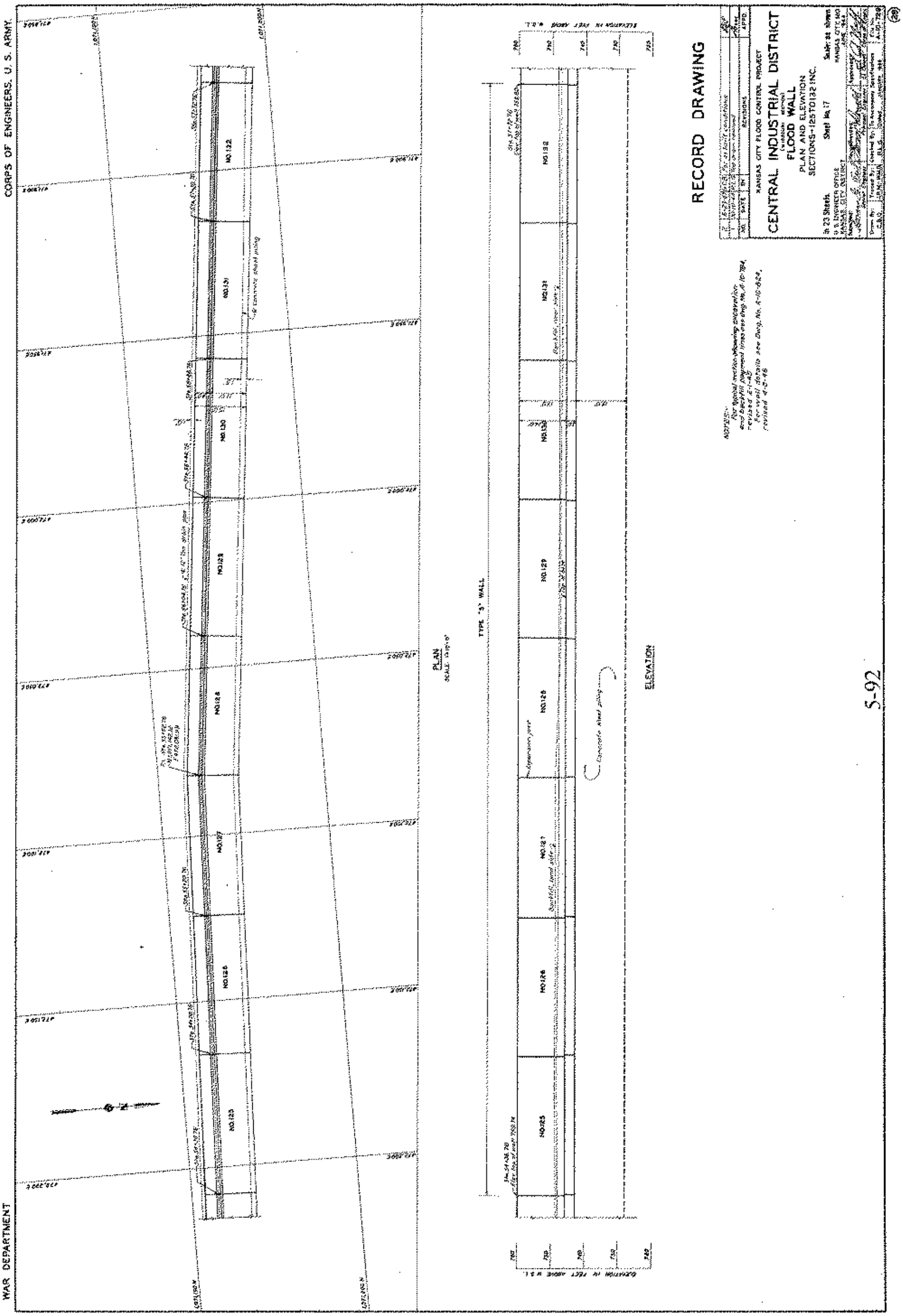


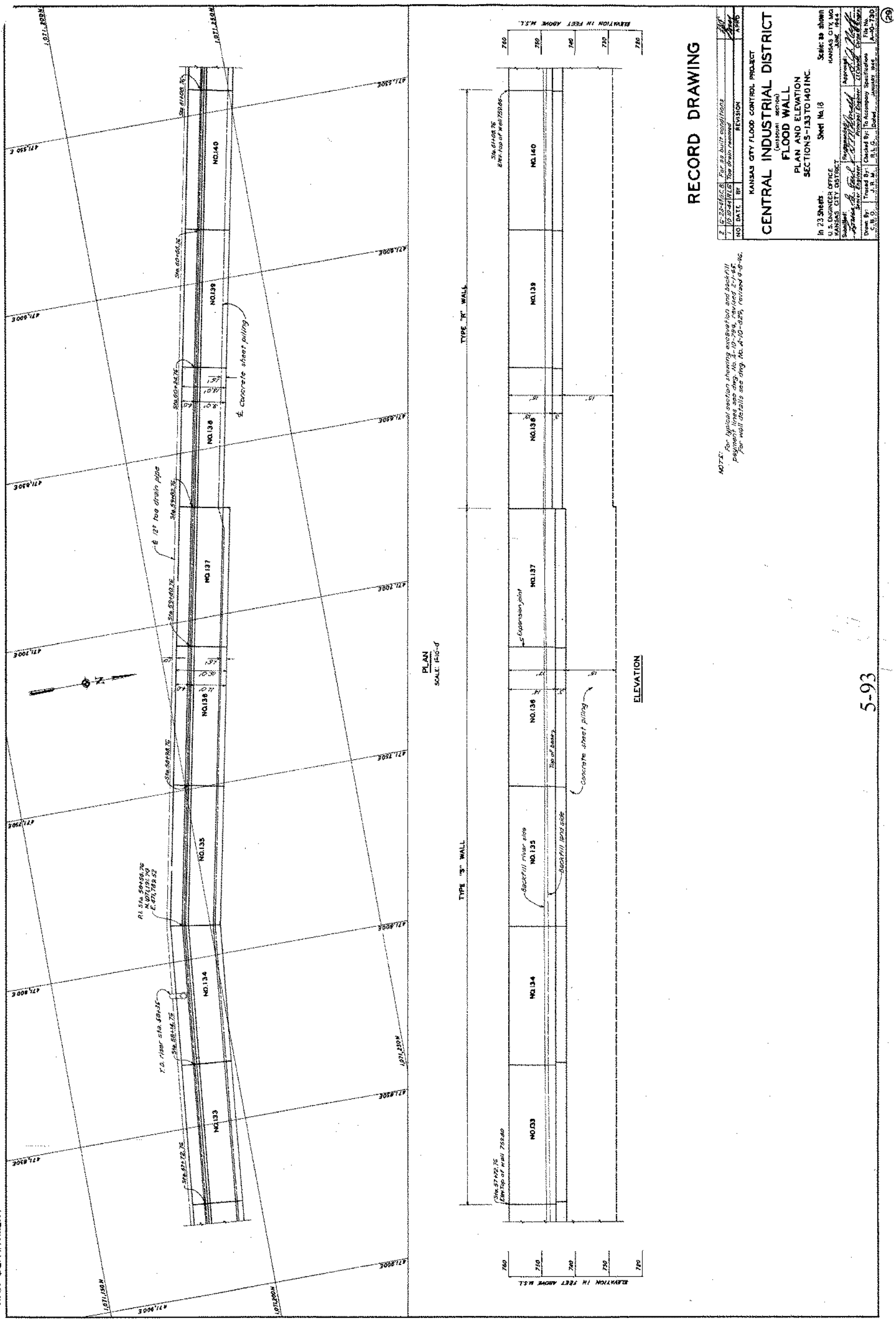
CORPS OF ENGINEERS, U. S. ARMY.
WAR DEPARTMENT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS–93 TO 100 INC.
Sheet No. 13
Scale: as shown
PLAN
ELEVATION
TYPE "N" WALL
TYPE "P" WALL
TYPE "Q" WALL
NO. 93
NO. 94
NO. 95
NO. 96
NO. 97
NO. 98
NO. 99
NO. 100
Concrete sheet piling
Cut & plugged
5-88


CORPS OF ENGINEERS, U. S. ARMY.
WAR DEPARTMENT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-101 TO 108 INC.
PLAN
TYPE "O" WALL
ELEVATION
5-89

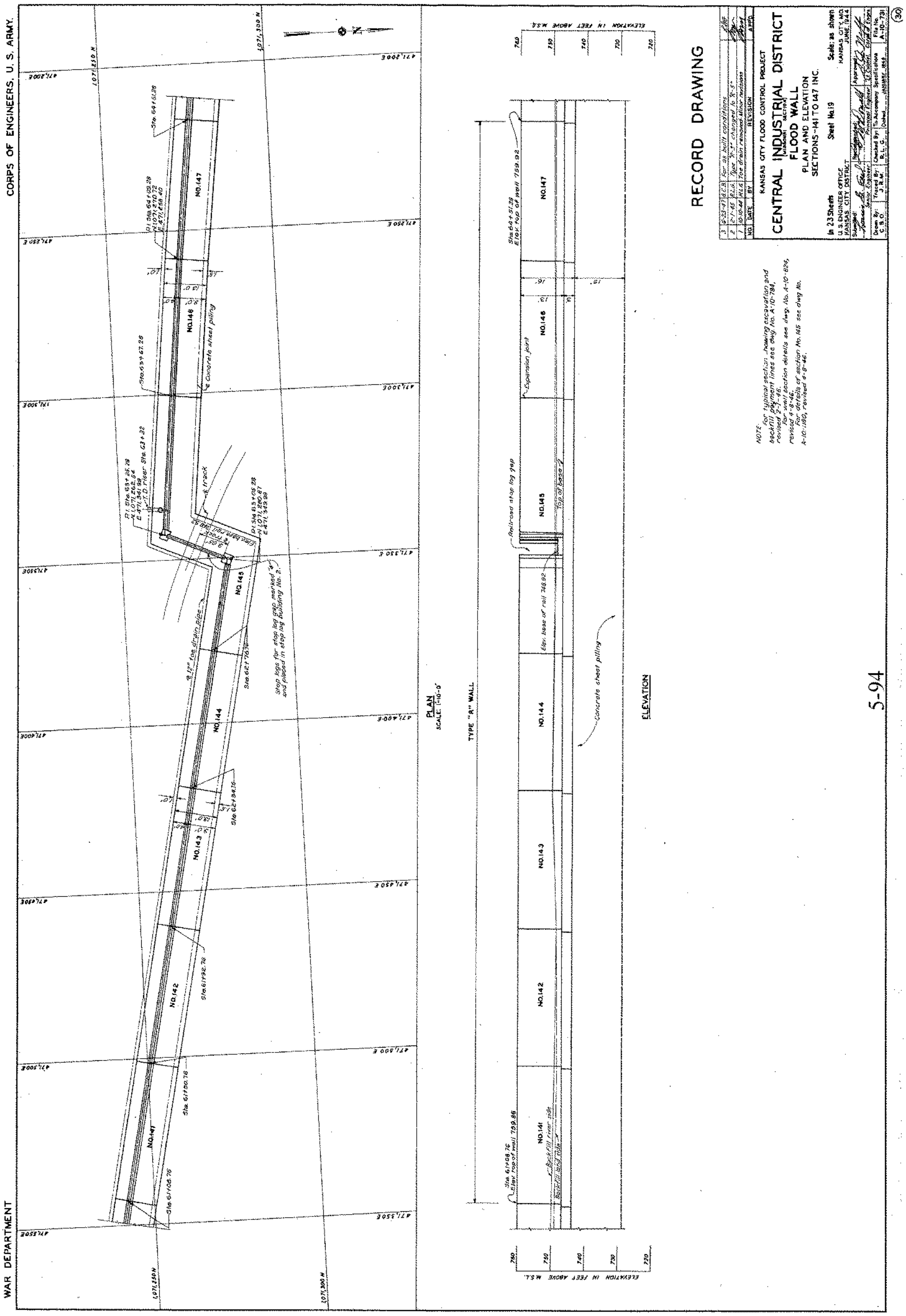

CORPS OF ENGINEERS, U. S. ARMY.
WAR DEPARTMENT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
(MISSOURI SECTION)
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-109 TO 118 INC.
In 23 Sheets
Sheet No. 15
U. S. ENGINEER OFFICE
KANSAS CITY DISTRICT
KANSAS CITY, MO.
PLAN
ELEVATION
TYPE "R" WALL
TYPE "Q" WALL
AMBROSE WINE CO. BLDG. NO. 2
Concrete sheet piling
ELEVATION IN FEET ABOVE M.S.L.
NO. 109
NO. 110
NO. 111
NO. 112
NO. 113
NO. 114
NO. 115
NO. 118
5-90

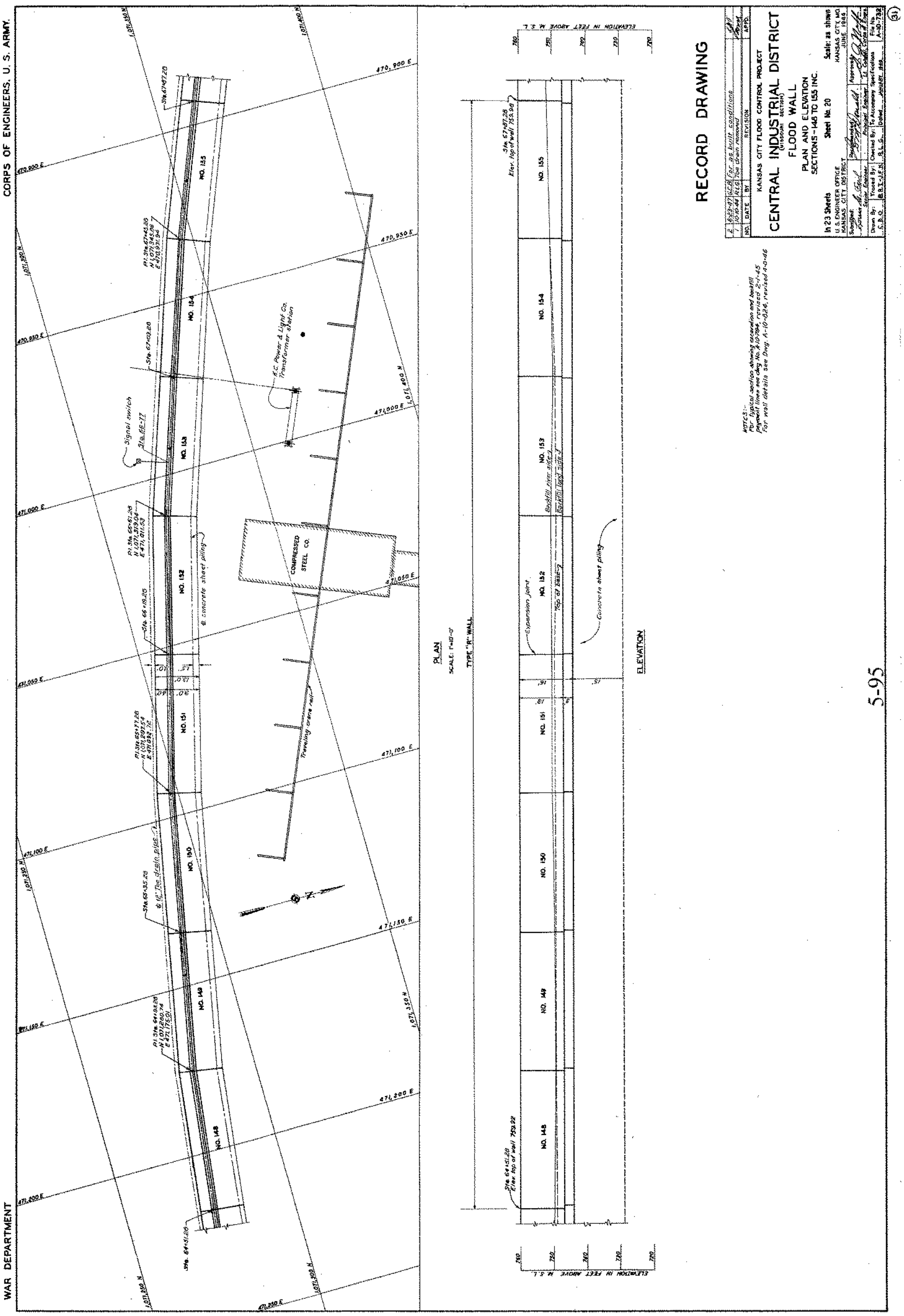

CORPS OF ENGINEERS, U. S. ARMY.
WAR DEPARTMENT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-117 TO 124 INC.
PLAN
ELEVATION
TYPE "B" WALL
PUMP HOUSE
NO.117
NO.118
NO.119
NO.120
NO.120A
NO.121
NO.122
NO.123
NO.124
5-91

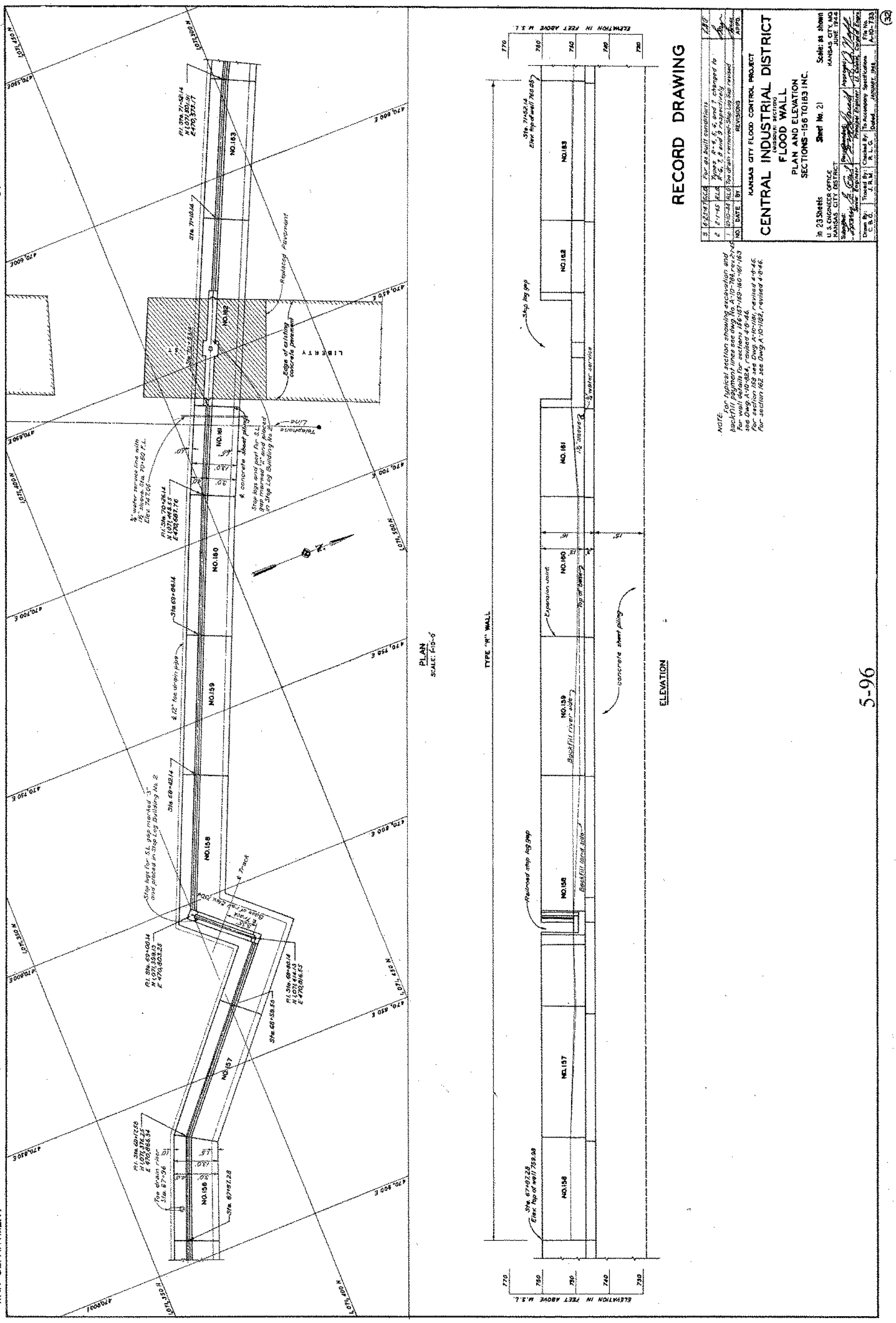

CORPS OF ENGINEERS, U. S. ARMY
WAR DEPARTMENT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS 125 TO 132 INC.
PLAN
TYPE "S" WALL
ELEVATION
NO.125
NO.126
NO.127
NO.128
NO.129
NO.130
NO.131
NO.132
5-92

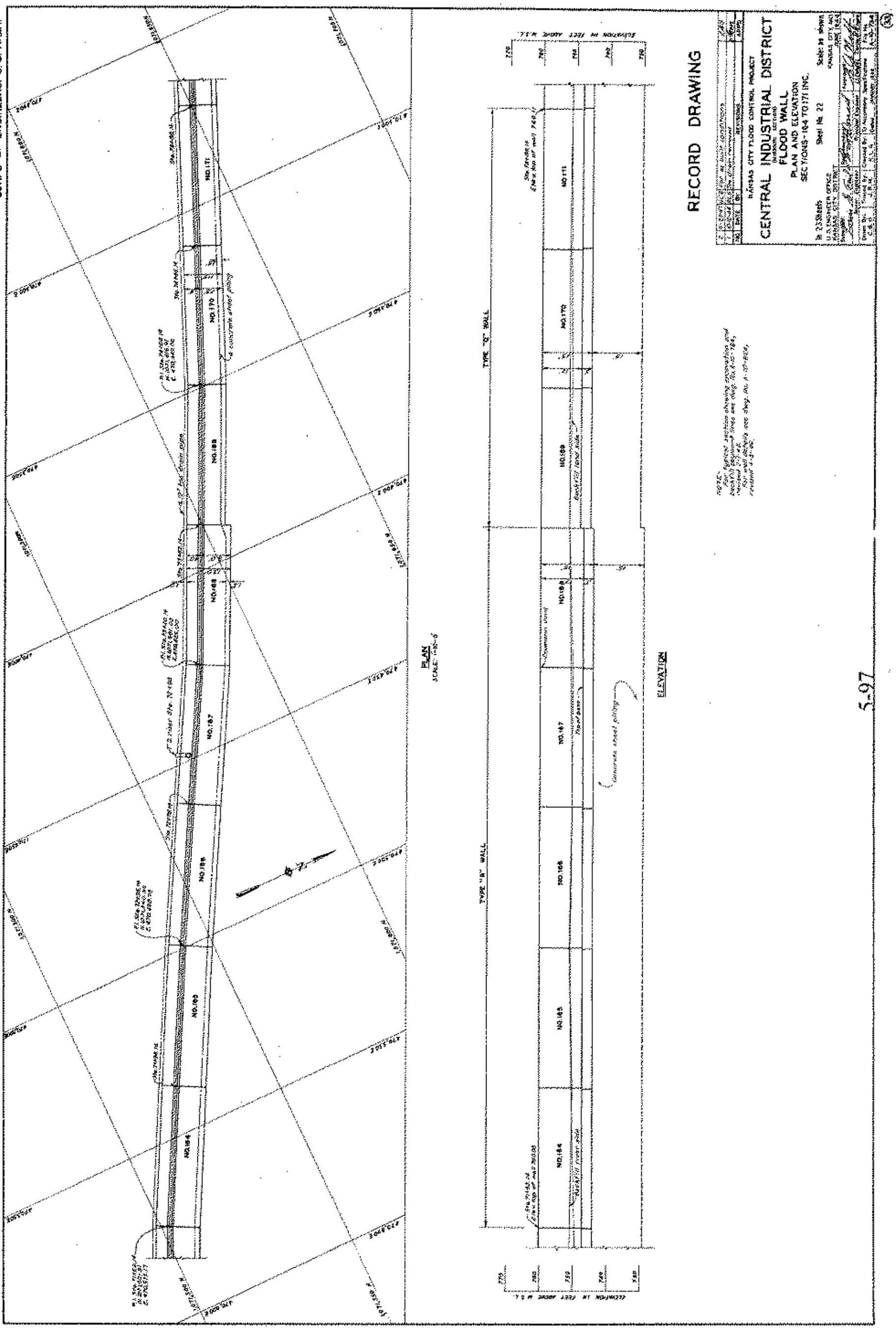


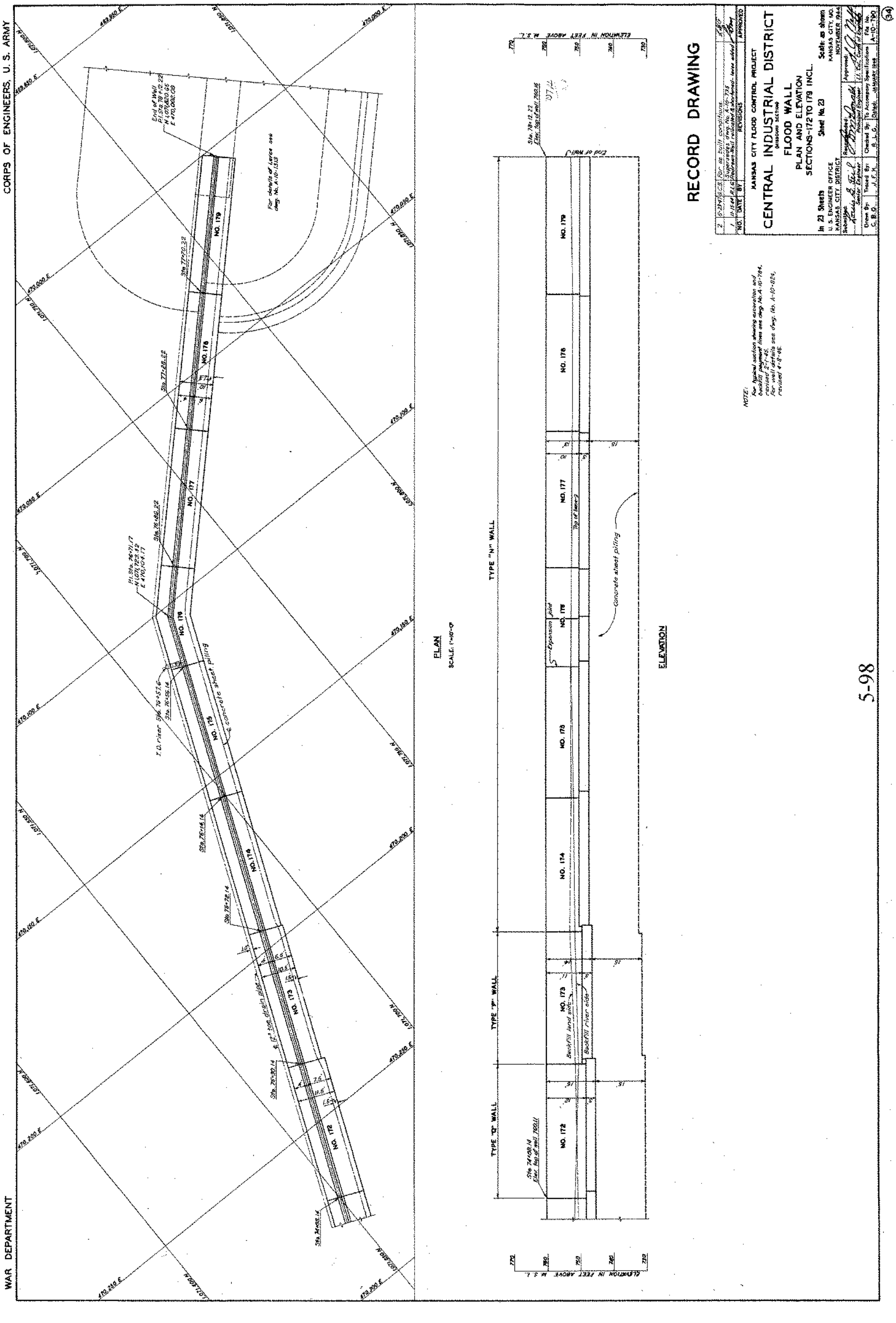
CORPS OF ENGINEERS, U. S. A.
WAR DEPARTMENT
PLAN
SCALE: 1"=10'-0"
ELEVATION
TYPE "S" WALL
TYPE "R" WALL
ELEVATION IN FEET ABOVE M.S.L.
NO.133
NO.134
NO.135
NO.136
NO.137
NO.138
NO.139
NO.140
Concrete sheet piling
Expansion joint
Backfill river side
Backfill land side
Top of base
NOTE: For typical section showing excavation and backfill payment lines see dwg. No. A-10-794, revised 2-1-45. For wall details see dwg. No. A-10-829, revised 4-8-46.
RECORD DRAWING
2 6-23-47 C.C.B. For as built conditions
1 10-10-44 R.L.G. Toe drain removed
NO DATE BY REVISION APPD
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
(MISSOURI SECTION)
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-133 TO 140 INC.
In 23 Sheets
Sheet No.18
Scale: as shown
U.S. ENGINEER OFFICE
KANSAS CITY DISTRICT
KANSAS CITY, MO.
JUNE 1944
Submitted: Senior Engineer
Recommended: Principal Engineer
Approved: Corps of Engrs.
Drawn By: C.B.O.
Traced By: J.R.M.
Checked By: R.L.G.
To Accompany Specifications Dated JANUARY 1944
File No. A-10-730
29
5-93

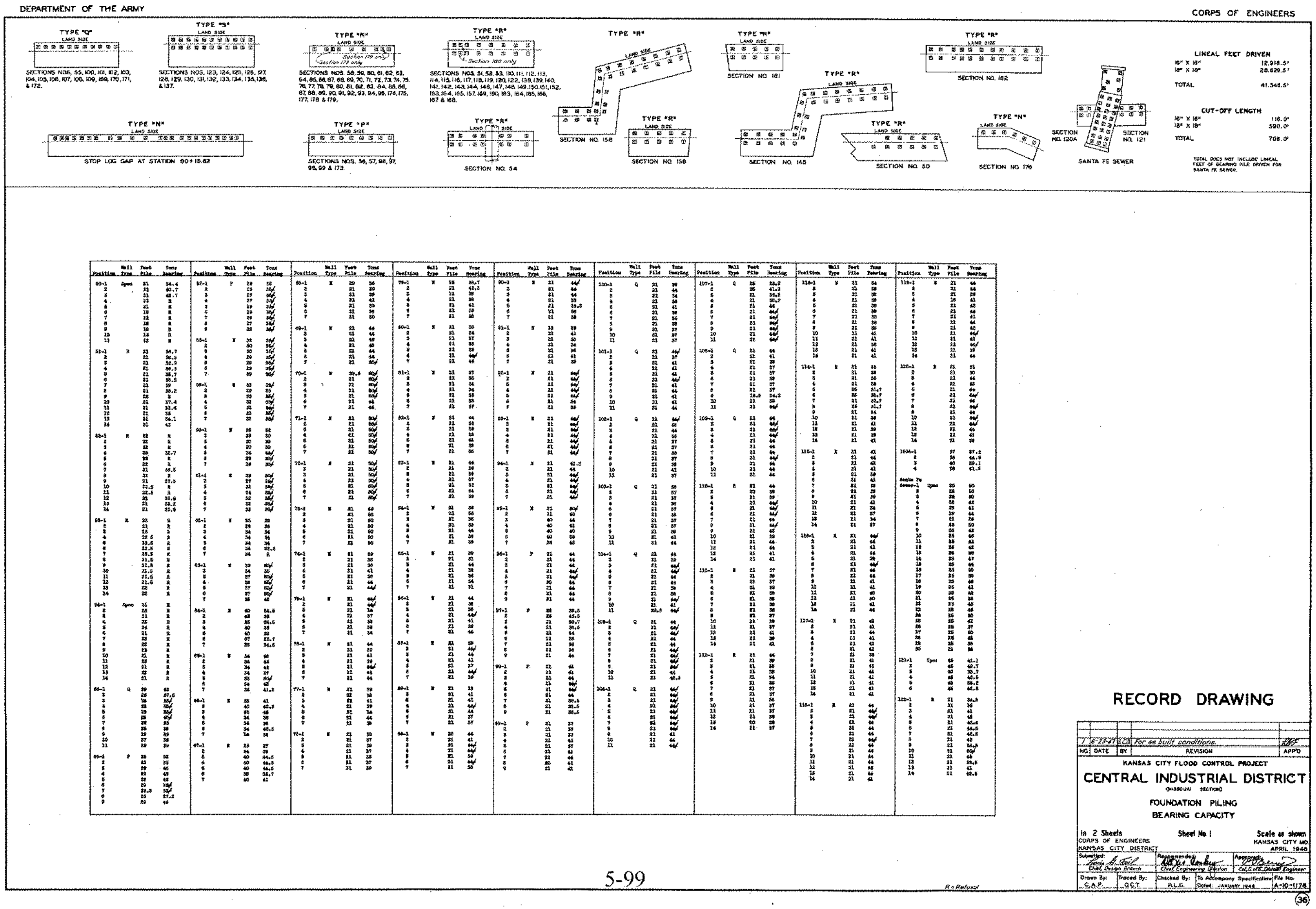

CORPS OF ENGINEERS, U. S. ARMY.
WAR DEPARTMENT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-141 TO 147 INC.
PLAN
TYPE "R" WALL
ELEVATION
NO.141
NO.142
NO.143
NO.144
NO.145
NO.146
NO.147
Concrete sheet piling
Expansion joint
Railroad stop log gap
5-94


CORPS OF ENGINEERS, U. S. ARMY.
WAR DEPARTMENT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS - 148 TO 155 INC.
Sheet No. 20
In 23 Sheets
Scale: as shown
U. S. ENGINEER OFFICE
KANSAS CITY DISTRICT
KANSAS CITY, MO.
JUNE 1944
PLAN
TYPE "R" WALL
ELEVATION
COMPRESSED STEEL CO.
K.C. Power & Light Co. Transformer station
Traveling crane rail
Signal switch
Concrete sheet piling
Expansion joint
ELEVATION IN FEET ABOVE M. S. L.
5-95


CORPS OF ENGINEERS. U. S. ARMY.
WAR DEPARTMENT
PLAN
TYPE "R" WALL
ELEVATION
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
(MISSOURI SECTION)
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-156 TO 183 INC.
5-96


CORPS OF ENGINEERS. U. S. ARMY.
WAR DEPARTMENT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-164 TO 171 INC.
Sheet No. 22
PLAN
ELEVATION
TYPE "A" WALL
TYPE "C" WALL
5.97


CORPS OF ENGINEERS, U. S. ARMY
WAR DEPARTMENT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
FLOOD WALL
PLAN AND ELEVATION
SECTIONS-172 TO 179 INCL.
PLAN
ELEVATION
TYPE "H" WALL
TYPE "P" WALL
TYPE "Q" WALL
NO. 172
NO. 173
NO. 174
NO. 175
NO. 176
NO. 177
NO. 178
NO. 179
5-98


DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
TYPE "Q"
LAND SIDE
SECTIONS NOS. 55, 100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 169, 170, 171, & 172.
TYPE "S"
SECTIONS NOS. 123, 124, 125, 126, 127, 128, 129, 130, 131, 132, 133, 134, 135, 136, & 137.
TYPE "N"
SECTIONS NOS. 58, 59, 60, 61, 62, 63, 64, 65, 66, 67, 68, 69, 70, 71, 72, 73, 74, 75, 76, 77, 78, 79, 80, 81, 82, 83, 84, 85, 86, 87, 88, 89, 90, 91, 92, 93, 94, 95, 174, 175, 177, 178 & 179.
TYPE "R"
SECTIONS NOS. 51, 52, 53, 110, 111, 112, 113, 114, 115, 116, 117, 118, 119, 120, 122, 138, 139, 140, 141, 142, 143, 144, 146, 147, 148, 149, 150, 151, 152, 153, 154, 155, 157, 159, 160, 163, 164, 165, 166, 167 & 168.
Section 180 only
TYPE "N"
STOP LOG GAP AT STATION 60+18.62
TYPE "P"
SECTIONS NOS. 56, 57, 96, 97, 98, 99 & 173.
SECTION NO. 54
SECTION NO. 158
SECTION NO. 156
SECTION NO. 181
SECTION NO. 145
SECTION NO. 50
SECTION NO. 182
SECTION NO. 176
SECTION NO. 120A
SECTION NO. 121
SANTA FE SEWER
LINEAL FEET DRIVEN
16" X 16" 12,916.5'
18" X 18" 28,629.5'
TOTAL 41,546.5'
CUT-OFF LENGTH
16" X 16" 118.0'
18" X 18" 590.0'
TOTAL 708.0'
TOTAL DOES NOT INCLUDE LINEAL FEET OF BEARING PILE DRIVEN FOR SANTA FE SEWER.
Position
Wall Type
Feet Pile
Tons Bearing
R = Refusal
RECORD DRAWING
NO. DATE BY REVISION APP'D
1 6-23-47 G.C.B. For as built conditions.
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
(MISSOURI SECTION)
FOUNDATION PILING
BEARING CAPACITY
In 2 Sheets
Sheet No. 1
Scale as shown
CORPS OF ENGINEERS
KANSAS CITY DISTRICT
KANSAS CITY MO
APRIL 1946
Submitted: Chief, Design Branch
Recommended: Chief, Engineering Division
Approved: Col. C of E, District Engineer
Drawn By: C.A.P.
Traced By: O.C.T.
Checked By: R.L.G.
To Accompany Specifications
Dated: JANUARY 1946
File No. A-10-1178
36

DEPARTMENT OF THE ARMY

CORPS OF ENGINEERS

Position	Wall Type	Feet Pile	Tons Bearing	Position	Wall Type	Feet Pile	Tons Bearing	Position	Wall Type	Feet Pile	Tons Bearing	Position	Wall Type	Feet Pile	Tons Bearing	Position	Wall Type	Feet Pile	Tons Bearing	Position	Wall Type	Feet Pile	Tons Bearing	Position	Wall Type	Feet Pile	Tons Bearing	Position	Wall Type	Feet Pile	Tons Bearing	Position	Wall Type	Feet Pile	Tons Bearing

RECORD DRAWING

NO.	DATE	BY	REVISION	APP'D
1	6-23-47	S.C.B.	For as built conditions.	[illegible]

KANSAS CITY FLOOD CONTROL PROJECT

CENTRAL INDUSTRIAL DISTRICT

(MISSOURI SECTION)

FOUNDATION PILING

BEARING CAPACITY

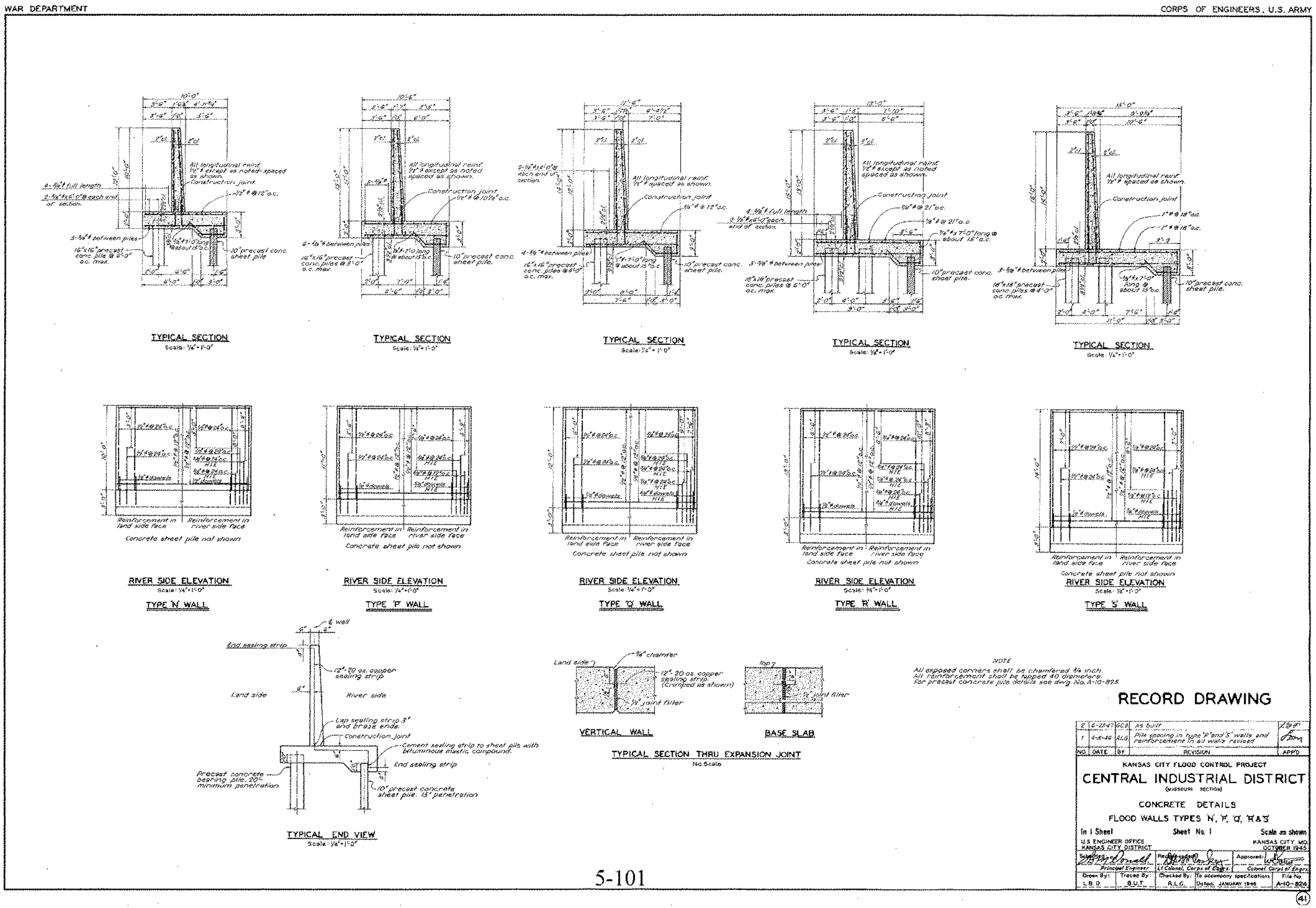
In 2 Sheets | Sheet No. 2 | Scale: as shown

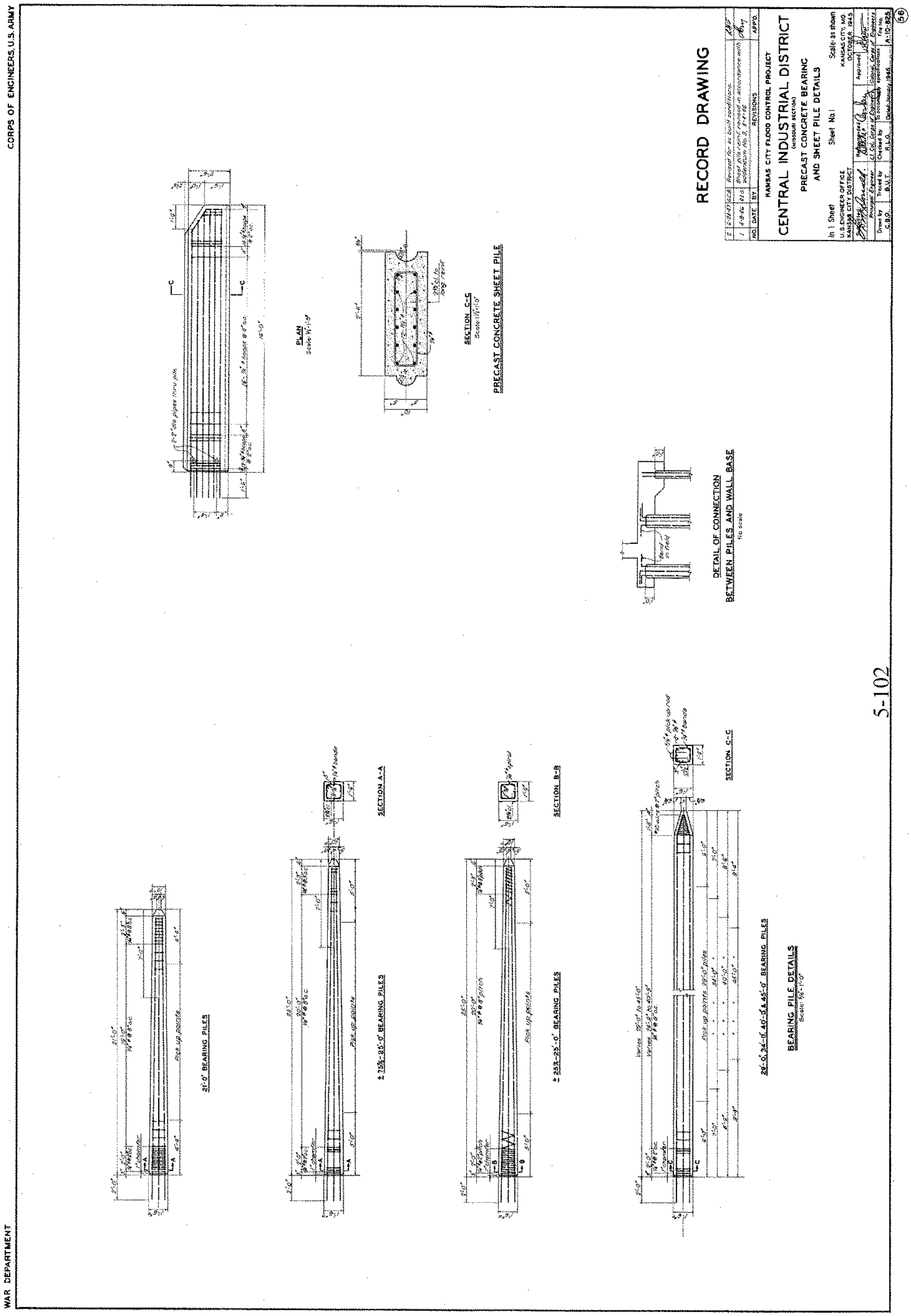
CORPS OF ENGINEERS, KANSAS CITY DISTRICT | KANSAS CITY, MO. APRIL 1946

Submitted: [signature] Chief, Design Branch | Recommended: [signature] Chief, Engineering Division | Approved: [signature] Col., C of E, District Engineer

Drawn By: C.A.P. | Traced By: O.C.T. | Checked By: R.L.G. | To Accompany Specifications Date JANUARY 1946 | File No. A-10-1179

(37)


WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
TYPICAL SECTION
RIVER SIDE ELEVATION
TYPE 'N' WALL
TYPE 'P' WALL
TYPE 'Q' WALL
TYPE 'R' WALL
TYPE 'S' WALL
TYPICAL END VIEW
VERTICAL WALL
BASE SLAB
TYPICAL SECTION THRU EXPANSION JOINT
No Scale
NOTE
All exposed corners shall be chamfered 3/4 inch.
All reinforcement shall be lapped 40 diameters.
For precast concrete pile details see dwg. No. A-10-825.
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
(MISSOURI SECTION)
CONCRETE DETAILS
FLOOD WALLS TYPES 'N', 'P', 'Q', 'R' & 'S'
In 1 Sheet
Sheet No. 1
Scale as shown
U.S. ENGINEER OFFICE
KANSAS CITY DISTRICT
KANSAS CITY, MO.
OCTOBER 1945
A-10-824
5-101



CORPS OF ENGINEERS, U.S. ARMY
WAR DEPARTMENT
RECORD DRAWING
KANSAS CITY FLOOD CONTROL PROJECT
CENTRAL INDUSTRIAL DISTRICT
(MISSOURI SECTION)
PRECAST CONCRETE BEARING
AND SHEET PILE DETAILS
In 1 Sheet
Sheet No. 1
Scale: as shown
U.S. ENGINEER OFFICE
KANSAS CITY DISTRICT
KANSAS CITY, MO.
OCTOBER 1945
REVISIONS
Principal Engineer
Lt. Col., Corps of Engineers
Colonel, Corps of Engineers
Drawn by C.B.O.
Traced by B.U.T.
Checked by R.L.O.
File No. A-10-825
56
PLAN
SECTION C-C
PRECAST CONCRETE SHEET PILE
DETAIL OF CONNECTION
BETWEEN PILES AND WALL BASE
No scale
21'-0" BEARING PILES
SECTION A-A
SECTION B-B
SECTION C-C
BEARING PILE DETAILS

EXHIBIT A-5.6

Limestone Friction Angle Determination

FROM: GENERAL DESIGN MEMORANDUM
BRUSH CREEK ENHANCED CHANNEL MODIFICATIONS
MAY 1990

measurements) tests were performed on representative undisturbed samples under confining chamber pressures ranging from 0.25 to 2.0 tsf. The general procedure involved tests on three specimens trimmed from the same horizon. Specimens were trimmed in a humidity controlled room to the dimensions of 1.4 inches in diameter by 3 inches in height. The test specimens were saturated with back pressures ranging from 3.46 to 5.54 tsf. Once saturated, the specimens were consolidated under the assigned confining pressure until the volume change versus log time plot indicated primary consolidation was complete. To reduce consolidation time, peripheral filter paper strips were placed on the specimen to accelerate drainage. The triaxial "R" tests were strain controlled at a rate of 0.012 percent per minute. Failure time ranged from 360 to 1,272 minutes. Overburden test results are shown on plates C-2 thru C-8.

b. Foundation Shales. Both unconfined compression and direct shear testing was performed on representative samples of the foundation shales. Classification and consistency limit tests were also performed on these samples. Direct shear tests were performed on the following shales: Wea, Fontana, Stark, and Galesburg. These tests consisted of shearing three intact specimens under normal loads ranging from 1 to 5 tsf. The specimens were sheared at a constant deformation rate of 0.0003 in/min. Foundation shale test results are shown on plates C-10 thru C-13.

c. Concrete/Rock Contact. Direct shear tests were performed to determine the strength of the concrete/rock contact for retaining walls and dams founded on limestone. These tests were performed on 3" x 3" specimens using standard soil testing equipment. A normal stress of 1.0 tsf was selected for all tests. This pressure roughly corresponds with the average pressure anticipated in the field. A total of four tests were performed, each representing a different potential field condition. Two tests represented the concrete/rock contact without bonding. These specimens were prepared by casting concrete against a natural bedding plane surface covered with plastic wrap. After the concrete set, the plastic wrap was removed and the pieces fit back together. One test was performed to quantify the bond strength. This was achieved by casting concrete directly against a sawcut rock surface. The final test was run on a pre-existing bedding plane break to determine the strength of the bedding planes within the upper Winterset limestone. Concrete/Rock contact test results are shown on plate C-14.

4. DESIGN VALUES. Items considered in the selection of design values include the number of tests, the nature of the material, and the location of the material with respect to potential failure surfaces.

a. Overburden. The foundation overburden varies from natural alluvium to fills. The majority of testing was performed on the fills since they are located in critical areas which could affect stability of the slopes or walls. Initial review of lab results seemed to indicate that material behind Troost wall had different strength parameters than the overburden in the remainder of the channel. Therefore, lab results were plotted separately to differentiate between overburden behind Troost wall (tests 4,5,6,7) and overburden in the remainder of the channel (tests 1,2,3). Test numbers 1,2, and 3 (UC-55, UC-57, and UC-59) indicate rather high cohesion values for both "R" and "S" envelopes. Further evaluation of data for these tests discovered irregularities in lab results and inconsistencies with torvane values and field description. Due to their suspect nature, these tests were not used in selection of overburden design parameters. The total stress strength parameters represent versus . The failure criteria was maximum deviator stress. This is different from the criteria used in Appendix D (Troost Wall Stability). The total stress failure criteria used for Troost Wall stability calculations was the maximum principle effective stress ratio. Effective stress strength parameters represent versus at the maximum principle effective stress ratio. Overburden test results are summarized on plate C-1.

(1) Total Stress ("R" strength): For overburden material in the Troost wall section a design strength of c=0.20 tsf, tan Ø=0.31 (Ø=17°) was selected.

(2) Effective Stress ("S" strength): For overburden material in the Troost wall section a design strength of c=0.05 tsf, tan Ø=0.42 (Ø=23°) was selected.

b. Foundation Shales. There were a total of ten direct shear tests run on the various shale members. A summary table for the shale testing is shown on plate C-9. The weakest shale members appear to be the Wea shale and the Galesburg shale. Peak shear strength parameters selected for these members are c=0.3 tsf and tan Ø=0.38 (Ø=21°). The most significant shale member for retaining wall design is the Fontana shale. The test performed on the Fontana shale (see plate C-11) indicates a peak shear strength of c=0.6 tsf, tan Ø=0.51 (Ø=27°) and a large strain shear strength of c=0, tan Ø=0.38 (Ø=21°). The unconfined compression test results are presented in Table C-1.

TABLE C-1

Unconfined Compression Tests

Wea Shale (Boring No. UC-96)

Depth El.	35.2-35.4	35.4-36.0
Dry Density (lbs/ft.^{3})	132.0	130.2
Moisture Content (%)	8.9	9.9
Unconfined Compressive Strength (TSF)	13.69	39.35

Fontant Shale (Boring No. UC-47A)

Depth El.	8.7-9.7
Dry Density (lbs./ft.^{3})	129.6
Moisture Content (%)	10.4
Unconfined Compressive Strength (TSF)	31.72

c. Concrete/Rock Contact. Listed below is a summary of test results on the concrete/rock contact tests. Laboratory test results can be found on plate C-14.

RVD 1/17/06
RSK

HOLE NO.	SAMPLE NO.	TEST TYPE	PEAK	LARGE STRAIN
C-100	1	Conc/Rock w/o bonding	54°	40°
C-100	2	Rock/Rock natural break	48°	39°
C-100	3	Conc/Shale w/ bonding	30° (c 2.5 tsf)	27°
C-100	8	Conc/Rock w/o bonding	42°	32°

Sample No.'s 1 & 8 tested concrete cast against a pre-existing bedding plane break with plastic wrap separating the two during casting. Sample No. 1 had larger asperities which resulted in a higher strength. Sample No. 2 tested a pre-existing bedding plane break pieced back together to investigate sliding resistance of the bedding planes within the upper Winterset limestone. Sample No. 3 tested a sawcut rock surface with concrete cast directly against the rock. This test was performed to quantify the bond strength and to determine the magnitude of sliding resistance derived from the asperities found on a normal rock surface. (Note: Common KCD practice is to ignore any bond strength for analysis purposes). Of the four tests performed, sample Nos. 1 & 2 are considered most representative of expected field conditions. Even so, they are believed to be a lower bound of what is possible in the field due to the following: (1) Samples selected for testing were those with the smallest asperities; (2) The plastic wrap used to prevent bonding between the concrete and rock specimens caused some bridging of the rock asperities which resulted in rounded rather than angular asperities in the concrete; (3) One would expect that both the number and size of bedding plane surface irregularities would be much greater over the area of a footing than found in a six-inch diameter core. Design strengths selected for both the Concrete/Rock contact and bedding planes in the upper Winterset are as follows:

Peak w/bonding	c=2.5 tsf
Peak w/o bonding	c=0.0 tsf, Ø = 45°
Large Strain	c=0.0 tsf, Ø = 35°

THESE VALUES ARE FOR THE WINTERSET LIMESTONE.

HOWEVER, the BETHANY FALLS @ CID-MO IS ≥ WINTERSET.

USE ϕ = ~~35°~~ FOR CID-MO

45 degrees

GMB 01/18/06

12/05/11

5-108

C-4

EXHIBIT A-5.7

Pile Capacity Reliability

CID-MO | Parameters for Risk/Relia. | To: EC-DS From: EC-GA

ϕ_{BL}

BELLEW
03/31/08

Expected value of $\bar{\phi}_{BLANKET}$ = 26°

Source: NKC Analysis of Design ... MAY 1945 it is stated that it most represents "actual" conditions.

COV $\bar{\phi}_{BLANKET}$ = 10%

SOURCE: ETL 1110-2-556

γ_{BL}

Expected value of γ_{BL} = 115 pcf

COV γ_{BL} = 5%

Source: ETL 1110-2-567 and ETL-2-556

Pile capacity

COV pile capacity = 25 comp.
18 tens.

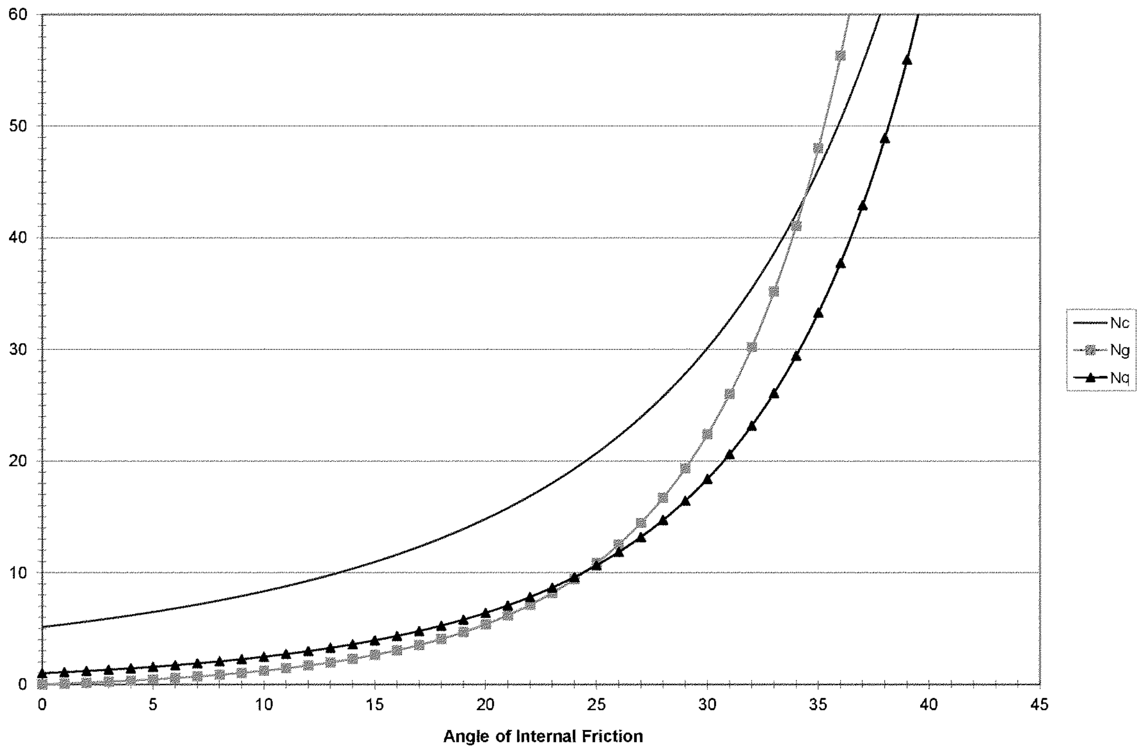
Source: ETL 1110-2-561

Note: there are other parameters in the reference material which may be used as desired.

EXHIBIT A-5.8

Floodwall Bearing Capacity

Chart Bearing Cap Coeff



Phi (degrees)	Phi (rads)	Nc	Nγ	Nq
0	0	5.14	0.0	1.0
1	0.017453293	5.4	0.1	1.1
2	0.034906585	5.6	0.2	1.2
3	0.052359878	5.9	0.2	1.3
4	0.06981317	6.2	0.3	1.4
5	0.087266463	6.5	0.4	1.6
6	0.104719755	6.8	0.6	1.7
7	0.122173048	7.2	0.7	1.9
8	0.13962634	7.5	0.9	2.1
9	0.157079633	7.9	1.0	2.3
10	0.174532925	8.3	1.2	2.5
11	0.191986218	8.8	1.4	2.7
12	0.20943951	9.3	1.7	3.0
13	0.226892803	9.8	2.0	3.3
14	0.244346095	10.4	2.3	3.6
15	0.261799388	11.0	2.6	3.9
16	0.27925268	11.6	3.1	4.3
17	0.296705973	12.3	3.5	4.8
18	0.314159265	13.1	4.1	5.3
19	0.331612558	13.9	4.7	5.8
20	0.34906585	14.8	5.4	6.4
21	0.366519143	15.8	6.2	7.1
22	0.383972435	16.9	7.1	7.8
23	0.401425728	18.0	8.2	8.7
24	0.41887902	19.3	9.4	9.6
25	0.436332313	20.7	10.9	10.7
26	0.453785606	22.3	12.5	11.9
27	0.471238898	23.9	14.5	13.2
28	0.488692191	25.8	16.7	14.7
29	0.506145483	27.9	19.3	16.4
30	0.523598776	30.1	22.4	18.4
31	0.541052068	32.7	26.0	20.6
32	0.558505361	35.5	30.2	23.2
33	0.575958653	38.6	35.2	26.1
34	0.593411946	42.2	41.1	29.4
35	0.610865238	46.1	48.0	33.3
36	0.628318531	50.6	56.3	37.8
37	0.645771823	55.6	66.2	42.9
38	0.663225116	61.4	78.0	48.9
39	0.680678408	67.9	92.2	56.0
40	0.698131701	75.3	109.4	64.2
41	0.715584993	83.9	130.2	73.9
42	0.733038286	93.7	155.5	85.4
43	0.750491578	105.1	186.5	99.0
44	0.767944871	118.4	224.6	115.3
45	0.785398163	133.9	271.7	134.9

Bearing Capacity Floodwalls

Material	
Cohesion psf	600
Phi Angle	0
Strip Foundation Footing Width B (ft)	10
Depth of Footing Df (ft)	5
Factor of Safety	1
Total Unit Weight pcf	115
Boyant Unit Weight pcf	52.6

Nc	5.1
Nγ	0.0
Nq	1.0

qult = c*Nc+.5*B*g'*Nγ+γt*Df+Nq

qult = 3659.00

qall = 3659

Bearing Capacity CID-MO Floodwalls

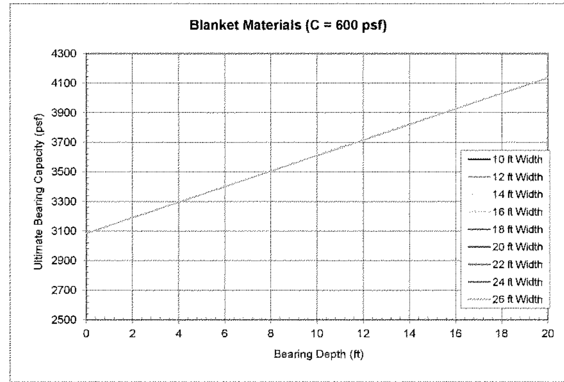
Material	Blanket
Cohesion psf	600
Phi Angle	0
Total Unit Weight pcf	115
Boyant Unit Weight pcf	52.6

Nc	5.1
Nγ	0.0
Nq	1.0

qult = c*Nc+.5*B*g'*Nγ+γt*Df+Nq

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
10	0	3084
10	2	3189
10	3	3242
10	4	3294
10	5	3347
10	6	3400
10	7	3452
10	8	3505
10	9	3557
10	10	3610
10	11	3663
10	12	3715
10	13	3768
10	14	3820
10	15	3873
10	16	3926
10	17	3978
10	18	4031
10	19	4083
10	20	4136

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
12	0	3084
12	2	3189
12	3	3242
12	4	3294
12	5	3347
12	6	3400
12	7	3452
12	8	3505
12	9	3557
12	10	3610
12	11	3663
12	12	3715
12	13	3768
12	14	3820
12	15	3873
12	16	3926
12	17	3978
12	18	4031
12	19	4083
12	20	4136



Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
14	0	3084
14	2	3189
14	3	3242
14	4	3294
14	5	3347
14	6	3400
14	7	3452
14	8	3505
14	9	3557
14	10	3610
14	11	3663
14	12	3715
14	13	3768
14	14	3820
14	15	3873
14	16	3926
14	17	3978
14	18	4031
14	19	4083
14	20	4136

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
16	0	3084
16	2	3189
16	3	3242
16	4	3294
16	5	3347
16	6	3400
16	7	3452
16	8	3505
16	9	3557
16	10	3610
16	11	3663
16	12	3715
16	13	3768
16	14	3820
16	15	3873
16	16	3926
16	17	3978
16	18	4031
16	19	4083
16	20	4136

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
18	0	3084
18	2	3189
18	3	3242
18	4	3294
18	5	3347

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
20	0	3084
20	2	3189
20	3	3242
20	4	3294
20	5	3347

18	6	3400
18	7	3452
18	8	3505
18	9	3557
18	10	3610
18	11	3663
18	12	3715
18	13	3768
18	14	3820
18	15	3873
18	16	3926
18	17	3978
18	18	4031
18	19	4083
18	20	4136

20	6	3400
20	7	3452
20	8	3505
20	9	3557
20	10	3610
20	11	3663
20	12	3715
20	13	3768
20	14	3820
20	15	3873
20	16	3926
20	17	3978
20	18	4031
20	19	4083
20	20	4136

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
22	0	3084
22	2	3189
22	3	3242
22	4	3294
22	5	3347
22	6	3400
22	7	3452
22	8	3505
22	9	3557
22	10	3610
22	11	3663
22	12	3715
22	13	3768
22	14	3820
22	15	3873
22	16	3926
22	17	3978
22	18	4031
22	19	4083
22	20	4136

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
24	0	3084
24	2	3189
24	3	3242
24	4	3294
24	5	3347
24	6	3400
24	7	3452
24	8	3505
24	9	3557
24	10	3610
24	11	3663
24	12	3715
24	13	3768
24	14	3820
24	15	3873
24	16	3926
24	17	3978
24	18	4031
24	19	4083
24	20	4136

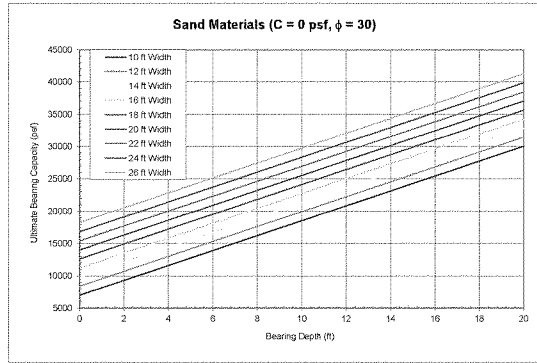
Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
26	0	3084
26	2	3189
26	3	3242
26	4	3294
26	5	3347
26	6	3400
26	7	3452
26	8	3505
26	9	3557
26	10	3610
26	11	3663
26	12	3715
26	13	3768
26	14	3820
26	15	3873
26	16	3926
26	17	3978
26	18	4031
26	19	4083
26	20	4136

Bearing Capacity CID-MO Floodwalls

Material	Sand
Cohesion psf	0
Phi Angle	30
Total Unit Weight pcf	125
Boyant Unit Weight pcf	62.6

Nc	30.1
Nγ	22.4
Nq	18.4

qult = c*Nc+.5*B*g'*Nγ+γt*Df+Nq



Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
10	0	7012
10	2	9316
10	3	10468
10	4	11620
10	5	12772
10	6	13923
10	7	15075
10	8	16227
10	9	17379
10	10	18531
10	11	19683
10	12	20835
10	13	21987
10	14	23139
10	15	24291
10	16	25443
10	17	26594
10	18	27746
10	19	28898
10	20	30050

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
12	0	8414
12	2	10718
12	3	11870
12	4	13022
12	5	14174
12	6	15326
12	7	16478
12	8	17630
12	9	18782
12	10	19933
12	11	21085
12	12	22237
12	13	23389
12	14	24541
12	15	25693
12	16	26845
12	17	27997
12	18	29149
12	19	30301
12	20	31453

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
14	0	9817
14	2	12121
14	3	13273
14	4	14424
14	5	15576
14	6	16728
14	7	17880
14	8	19032
14	9	20184
14	10	21336
14	11	22488
14	12	23640
14	13	24792
14	14	25944
14	15	27095
14	16	28247
14	17	29399
14	18	30551
14	19	31703
14	20	32855

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
16	0	11219
16	2	13523
16	3	14675
16	4	15827
16	5	16979
16	6	18131
16	7	19283
16	8	20434
16	9	21586
16	10	22738
16	11	23890
16	12	25042
16	13	26194
16	14	27346
16	15	28498
16	16	29650
16	17	30802
16	18	31954
16	19	33105
16	20	34257

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
18	0	12622
18	2	14925
18	3	16077
18	4	17229
18	5	18381
18	6	19533
18	7	20685
18	8	21837

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
20	0	14024
20	2	16328
20	3	17480
20	4	18632
20	5	19784
20	6	20935
20	7	22087
20	8	23239

18	9	22989
18	10	24141
18	11	25293
18	12	26444
18	13	27596
18	14	28748
18	15	29900
18	16	31052
18	17	32204
18	18	33356
18	19	34508
18	20	35660

20	9	24391
20	10	25543
20	11	26695
20	12	27847
20	13	28999
20	14	30151
20	15	31303
20	16	32455
20	17	33606
20	18	34758
20	19	35910
20	20	37062

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
22	0	15426
22	2	17730
22	3	18882
22	4	20034
22	5	21186
22	6	22338
22	7	23490
22	8	24642
22	9	25794
22	10	26945
22	11	28097
22	12	29249
22	13	30401
22	14	31553
22	15	32705
22	16	33857
22	17	35009
22	18	36161
22	19	37313
22	20	38465

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
24	0	16829
24	2	19133
24	3	20284
24	4	21436
24	5	22588
24	6	23740
24	7	24892
24	8	26044
24	9	27196
24	10	28348
24	11	29500
24	12	30652
24	13	31804
24	14	32955
24	15	34107
24	16	35259
24	17	36411
24	18	37563
24	19	38715
24	20	39867

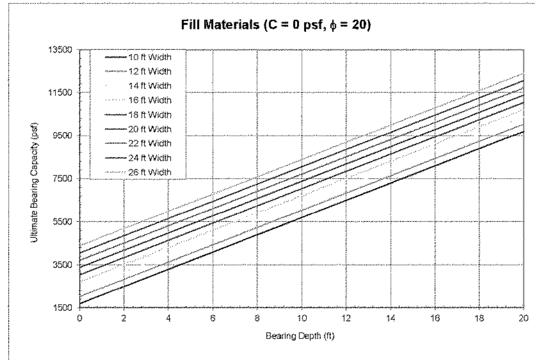
Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
26	0	18231
26	2	20535
26	3	21687
26	4	22839
26	5	23991
26	6	25143
26	7	26295
26	8	27446
26	9	28598
26	10	29750
26	11	30902
26	12	32054
26	13	33206
26	14	34358
26	15	35510
26	16	36662
26	17	37814
26	18	38966
26	19	40117
26	20	41269

Bearing Capacity CID-MO Floodwalls

Material	Fill
Cohesion psf	0
Phi Angle	20
Total Unit Weight pcf	125
Boyant Unit Weight pcf	62.6

Nc	14.8
Nγ	5.4
Nq	6.4

qult = c*Nc+.5*B*g'*Nγ+γt*Df+Nq



Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
10	0	1686
10	2	2487
10	3	2888
10	4	3288
10	5	3689
10	6	4090
10	7	4490
10	8	4891
10	9	5291
10	10	5692
10	11	6093
10	12	6493
10	13	6894
10	14	7294
10	15	7695
10	16	8096
10	17	8496
10	18	8897
10	19	9297
10	20	9698

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
12	0	2023
12	2	2824
12	3	3225
12	4	3626
12	5	4026
12	6	4427
12	7	4827
12	8	5228
12	9	5629
12	10	6029
12	11	6430
12	12	6830
12	13	7231
12	14	7632
12	15	8032
12	16	8433
12	17	8833
12	18	9234
12	19	9635
12	20	10035

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
14	0	2360
14	2	3161
14	3	3562
14	4	3963
14	5	4363
14	6	4764
14	7	5164
14	8	5565
14	9	5966
14	10	6366
14	11	6767
14	12	7168
14	13	7568
14	14	7969
14	15	8369
14	16	8770
14	17	9171
14	18	9571
14	19	9972
14	20	10372

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
16	0	2697
16	2	3499
16	3	3899
16	4	4300
16	5	4700
16	6	5101
16	7	5502
16	8	5902
16	9	6303
16	10	6703
16	11	7104
16	12	7505
16	13	7905
16	14	8306
16	15	8706
16	16	9107
16	17	9508
16	18	9908
16	19	10309
16	20	10710

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
18	0	3035
18	2	3836
18	3	4236
18	4	4637
18	5	5038
18	6	5438
18	7	5839
18	8	6239

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
20	0	3372
20	2	4173
20	3	4574
20	4	4974
20	5	5375
20	6	5775
20	7	6176
20	8	6577

18	9	6640
18	10	7041
18	11	7441
18	12	7842
18	13	8242
18	14	8643
18	15	9044
18	16	9444
18	17	9845
18	18	10245
18	19	10646
18	20	11047

20	9	6977
20	10	7378
20	11	7778
20	12	8179
20	13	8580
20	14	8980
20	15	9381
20	16	9781
20	17	10182
20	18	10583
20	19	10983
20	20	11384

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
22	0	3709
22	2	4510
22	3	4911
22	4	5311
22	5	5712
22	6	6113
22	7	6513
22	8	6914
22	9	7314
22	10	7715
22	11	8116
22	12	8516
22	13	8917
22	14	9317
22	15	9718
22	16	10119
22	17	10519
22	18	10920
22	19	11320
22	20	11721

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
24	0	4046
24	2	4847
24	3	5248
24	4	5649
24	5	6049
24	6	6450
24	7	6850
24	8	7251
24	9	7652
24	10	8052
24	11	8453
24	12	8853
24	13	9254
24	14	9655
24	15	10055
24	16	10456
24	17	10856
24	18	11257
24	19	11658
24	20	12058

Footing Width (ft)	Bearing Depth (ft)	Ultimate Bearing Capacity (psf)
26	0	4383
26	2	5185
26	3	5585
26	4	5986
26	5	6386
26	6	6787
26	7	7188
26	8	7588
26	9	7989
26	10	8389
26	11	8790
26	12	9191
26	13	9591
26	14	9992
26	15	10392
26	16	10793
26	17	11194
26	18	11594
26	19	11995
26	20	12395

Kansas Citys, Missouri and Kansas
Flood Risk Management Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Final Feasibility Report

Chapter A-6

CIVIL DESIGN
ARMOURDALE

CHAPTER A-6
CIVIL DESIGN - ARMOURDALE

A-6.1 SITE SELECTION AND PROJECT DEVELOPMENT

A-6.1.1 Introduction

This chapter of the engineering appendix presents the results of the civil design evaluation performed as part of the future conditions analysis for the Armourdale Unit of the Kansas Citys, Missouri and Kansas, Flood Risk Management Project. The U.S. Army Corps of Engineers (USACE), Kansas City District designed and constructed the Kansas Citys protection system. This portion of the study considers raises on the Armourdale Unit to the 0.2% (500-year), 0.2% (500-year)-plus-3-ft, and the 0.2% (500-year)-plus-5-ft water surface profile elevations, hereafter referred to as N500+0, N500+3, and N500+5 elevations.

The Armourdale Unit is located in Wyandotte County, Kansas, along the left bank of the Kansas River from mile 6.7 (Mattoon Creek) to mile 0.3, near the junction of the Kansas River with the Missouri River. The flood protection unit includes levees, floodwalls, riprap and levee toe protection, toe drains along the concrete floodwalls, surfaced levee crown, ramps and turnouts, seeded landside slopes of levees, stop log gaps, sandbag gaps, drainage structures, relief wells, and pumping plants. Stationing of the unit consists of the upper end (UE) extension, the lower end (LE) extension, and the area in between. The unit begins at Station 0+05 UE and ends at Station 61+00 LE. Stationing is summarized below along with station equations:

- 0+05 UE to 20+08.89 (Station Equation 20+08.89 UE BK = 9+71.16 AH)
- 9+71.16 to 206+12.43 BK (Station Equation 206+12.43 BK = 212+00 AH)
- 212+00 to 257+66.26 (Station Equation 257+66.26 BK = 257+64.97 AH)
- 257+64.97 to 322+85.41 (Station Equation 322+85.41 BK = 39+71.83 LE AH)
- 39+71.83 LE to 61+00 LE (N500+3 will end at Station 42+50 LE)

A-6.1.2 Levee Footprint

The unit includes sections of levee and floodwall. All alternatives include widening the levee footprint, modifying floodwall, replacing floodwalls or placing a floodwall on top of existing levee. Stability berms or underseepage features are needed as indicated by the geotechnical analysis.

A-6.1.3 Borrow Area

Prospective borrow areas were identified by the Sponsor and screened through joint Corps and Sponsor efforts during the Phase 1 portion of this feasibility project. Regarding borrow area, Phase 1 focused on the Argentine Unit. The borrow area for the

Armourdale Unit will be the same as for Argentine. Refer to Exhibit A-6.1 titled, "Borrow Area for Proposed Armourdale Unit Raise", for a discussion of the borrow area for the project at the end of this chapter.

A-6.1.4 Haul Routes

Haul routes from the borrow site at WaterOne to various points along the levee/floodwall alignment were generated based on the limited access points to the levee/floodwall system along the Armourdale Unit. Exhibit A-6.2, at the end of this chapter, shows the haul routes and their distances from WaterOne to various access points along the Armourdale Unit.

A-6.2 REAL ESTATE CONSIDERATIONS

Research is being conducted by CENWK Real Estate Staff to complete Preliminary Attorney's Opinion of Compensability as utilities are identified. Any conclusion or categorization that an item is a utility or facility relocation would result in work to be performed at the cost of the non-federal sponsor as part of LERRD responsibilities and is preliminary only. During PED, the Government will make a final determination of the relocations necessary for the construction, operation or maintenances of the project after further analysis and completion and approval of Final Attorney's Opinions of Compensability for each of the impacted utilities and facilities. Further detail on all real estate issues, is discussed in the Real Estate Plan, Appendix C to the Feasibility Report. Also refer to the real estate plan for a discussion of real estate issues pertaining to borrow area.

A-6.3 UTILITY RELOCATIONS

A review of the Kansas City District's criteria for utility lines was performed. Based on discussions, a criteria document specific to this project was developed, and is shown as Exhibit A-6.3,"Kansas City's Levee Gravity and Utility Pipeline Guidance" at the end of this chapter. This document was used in determining the disposition of existing utility lines crossing the Armourdale Unit.

A-6.3.1 Utility Levee Crossings

The study of utilities crossing the Armourdale Unit was conducted to estimate costs for relocation or removal of functioning or abandoned utilities. Using the criteria indicated above, it was determined that most pressure pipelines currently passing under the levee would be relocated over the levee. Refer to the Maps section of the Feasibility Report for the locations. Refer to Exhibit A-6.4 titled "Armourdale Utility Crossings: Inventory and Action for N500+3 Raise", located in the Supplemental Exhibits section, for the recommended action for each utility crossing the levee. Some utilities are to remain in place, others will be relocated up and over the levee. The drawing in Exhibit A-6.5, located in the Supplemental Exhibits section, displays typical utility crossings.

Utility crossing recommendations are based on the N500+3 raise, therefore the quantities for the cost estimate are also based on the N500+3 raise. For the N500+0 and N500+5 alternatives, recommendations for utility crossings are the same as for N500+3. Cost estimates and quantities used for determining the N500+3 cost were used for the N500+0 and the N500+5. There is negligible difference between the three raises in regards to recommended utility crossings. This approach for estimating the N500+0 and the N500+5 raise is similar to the approach taken in Phase 1 when cost estimating the Argentine Unit utility crossings.

A-6.3.2 Special Design and Construction Considerations

The project team will conduct specific utilities relocation coordination and design planning prior to levee raise construction contract award. In recent projects, this relocation work has proven very problematic if not thoroughly scheduled and coordinated. Sponsors, and utility owners, are responsible for most utility relocations (for those utilities deemed without legal compensability), the Kansas City District must be consulted for approval of the relocation design and schedule. Detailed planning for utility relocations and assignment of responsibilities is fully developed by the latter stages of the PED phase. All parties (sponsor, utility owner, and Corps of Engineers) must prepare for a highly coordinated utility relocation effort as the levee raise begins.

Where lines are shown as abandoned or to be abandoned, the Kansas City's Levee and Floodwall Utility Crossings Criteria will be followed.

A-6.3.3 Power Lines

Several large capacity power lines cross the Armourdale Unit. Most of the lines that cross the levee also cross the Kansas River and therefore have substantial structures holding them in place. These structures that interfere with the proposed levee raise will be avoided with the use of T-wall on levee or retaining walls when levee raises would extend the footprint. Following is a list of the stations currently crossed by power lines:

- **Station 74+00** – no apparent interference observed in field

- **Station 92+00** – levee raise or berm would interfere with power line support structure but the proposed T-wall on levee will not

- **Station 110+00** - levee raise or berm would interfere with power line support structure; T-wall on levee would not. Overhead power lines are located near the transition between levee and T-wall on levee, therefore this interference will need to be determined during detailed design when a land survey is completed.

- **Station 156+00** – no apparent interference observed in field as Mill Street pump plant is located between power line support structure and levee unit

- **Station 184+00** – levee raise or berm may interfere with power line support structure; this interference will need to be determined during detailed design when a land survey is completed.

- **Station 196+00** – levee raise or berm would interfere with power line support structure but the proposed T-wall on levee will not

- **Station 228+00** - levee raise or berm may interfere with power line support structure but the proposed T-wall on levee will not

The existing clearance between most of the power lines and top of levee is approximately 40-ft although this will need to be verified by a land survey during detailed design phases. The N500+3 alternative results in a levee raise in the range of 3-ft to 5-ft. Coordination with the Board of Public Utilities (BPU) determined that the required clearance between the power lines and the levee is 20.9-ft. This clearance is based on the National Electric Safety Code (NESC). With the maximum raise of 5-ft reducing the minimum clearance to 35-ft, the clearance between the power lines and the levee is adequate.

Power lines running parallel (landward) with the levee will have to be protected during construction. Depending on proposed stability berm and relief wells, some small power lines may need to be relocated or protected during and after construction.

A-6.3.4 Utility Uplift

The study of uplift on existing utilities was conducted to estimate costs for relocation or removal of functioning or abandoned utilities. Regions were identified for utility uplift concern, based on geotechnical and N500+3 conditions. The region extends 500-ft landward of the levee/ floodwall centerline and corresponds with the "critical zone" of the levee/floodwall.

Water and sanitary sewer utility mapping was obtained from the Unified Government of Wyandotte County and the Board of Public Utilities. Storm sewers that do not cross the levee were not mapped because uplift is generally not a concern as long as both ends of the pipe are open to atmosphere. Being open to atmosphere at both ends allows the pipe to fill during inundation. Storm sewers are typically metal or concrete and heavy enough not to float when filled with water. For the purposes of uplift it was assumed that underground electrical lines (UGE) were not affected. The information on natural gas lines and petroleum lines within the 500-ft zone was limited but was evaluated where the information was available. No known petroleum lines are located within the critical zone. Natural gas and water service lines to buildings are generally less than 6 inches in diameter and not located near the levee. Based upon previous uplift calculations, lines 6 inches and smaller are generally not affected by uplift, therefore only lines 6 inches and greater have been evaluated.

For this study, the Armourdale Unit was broken into seven segments for analysis with each segment having its own geotechnical features. The geotechnical input consisted of impervious blanket thickness and foundation sands thickness as well as levee dimensions, berm dimensions, bedrock depth, and soil density. Exhibit A-6.6 "Armourdale Utility Uplift Spreadsheet: Data Entry Worksheet" displays the existing geotechnical and dimensional conditions for each levee segment at the end of this chapter. For this study, the driving head of water represents the N500+3 level of protection. The geotechnical designers provided the dissipation of the hydraulic gradient through the impervious blanket for use in calculating uplift on utilities. The locations of underseepage control features (cutoff walls) and wells were also considered in regards to uplift on utility lines.

Exhibit A-6.7 contains a sample calculation for utility uplift at the end of this chapter. It shows how each of the geotechnical, dimensional, and hydraulic gradient inputs are used to calculate potential uplift concerns. The utility uplift spreadsheets for levees are based on this sample calculation. A set of spreadsheets was developed for each levee segment for 6-inch, 12-inch, 16-inch, 24-inch and 48-inch pipes. Uplift was evaluated for each pipe size beginning at the toe of the levee. If uplift was not a concern at the toe of the levee at 40 inches deep, then no further evaluation was done on that pipe size and the result displayed on the "Uplift Concern" maps is that there is no uplift concern for that pipe size in that stretch of the levee. There are three uplift maps for Armourdale utilities: water (Exhibit A-6.8), sanitary sewer (Exhibit A-6.9), and natural gas (Exhibit A-6.10) located at the end of this chapter.

If uplift is found to be a concern at the toe of the levee at 40 inches deep, then uplift was evaluated further from the toe up to 500-ft from the toe of the levee. The distance from the levee at which the factor of safety equals 1.1 is the result displayed on the "Uplift Concern" maps.

The final product of the uplift spreadsheet analysis provided the limits from the centerline of the levee that a given type, size and depth of pipe must be located in order to meet the minimum uplift factor of safety. Exhibit A-6.11,"N500+3 Utility Uplift Calculations", located in the supplemental exhibits section, displays this analysis and is labeled for the various raise and levee segments, , the size and type of line, and the length of line to be lowered or covered to alleviate uplift concern

The areas of uplift concern were overlaid on the utilities map to develop maps of "Uplift Concern" (Exhibits A-6.8 through A-6.10). The uplift zones were used to estimate the total length of pipe that would have to be lowered or covered with additional material to eliminate the uplift concern.

Utility uplift recommendations are based on the N500+3 raise, therefore the quantities for the cost estimate is also based on the N500+3 raise. For the N500+0 and N500+5 alternatives, quantities for utility uplift are not the same as for N500+3. Quantities used for determining the N500+3 cost will be adjusted up for the N500+5 alternative and down for the N500+0 alternative. This approach for estimating the N500+0 and the N500+5 raise is similar to the approach taken in Phase 1 for estimating the Argentine Unit utility uplift recommendations. Based on Argentine quantities for uplift, the quantity of piping for the N500+0 raise will be calculated at 80% of the N500+3 quantity. The quantity of piping for the N500+5 raise will be calculated at 120% of the N500+3 quantity. The number of manholes to be replaced will remain the same for all alternatives, N500+0, N500+3, and N500+5.

A-6.3.5 Inspection Trench

Since the proposed construction primarily involves raising the existing protection, no specific locations for new inspection trenches are indicated or considered. In the event that conditions are encountered in the field which warrants investigation, an inspection trench may be used.

A-6.4 REFERENCES

1. American Water Works Association "Steel Pipe – A Guide for Design and Installation", AWWA M11 4, 2004.

2. Hydraulic Institute "Hydraulic Institute Engineering Data Book" Hydraulic Institute, Cleveland, Ohio.

3. EM 1110-2-1913, "Engineering and Design – Design and Construction of Levees"

4. Kansas City District regional specific guidance http://www.nwk.usace.army.mil/Missions/EngineeringDivision/GeotechnicalBranch/GeotechnicalDesignandDamSafety.aspx

A-6.5 SUPPLEMENTAL EXHIBITS

EXHIBIT A-6.1

Borrow Area for Proposed Armourdale Unit Raise

Borrow Area for Proposed Armourdale Unit Raise

Prospective borrow areas were identified by the Sponsor and screened through joint Corps and Sponsor efforts during the Phase I portion of this feasibility project. Regarding borrow area, Phase 1 focused on the Argentine Unit. Because the borrow area for Armourdale will be the same as for Argentine, this write-up was taken from the Phase I report and updated to include the Armourdale Unit raises.

ARGENTINE

Total required fill quantities for Argentine are 90,301, 257,881, and 508,281 compacted cubic yards (ccy) for N500, N500+3, and N500+5 raises, respectively. The N500+3 raise is the recommended alternative for Phase 1, therefore 258,000 ccy will be used for estimating borrow needs. For Argentine, the proposed levee raise accounts for about half of the fill requirement and stability or underseepage berms account for the other half. Subsurface investigation of the borrow area provided the required geotechnical information for the materials to be used in the levee.

ARMOURDALE

Total required borrow quantities from WaterOne for Armourdale is approximately 350,000 bank cubic yards (bcy) for the N500+3 raise. This quantity is based on balancing the cut and fill on site and then calculating what is required to supplement on-site material with that from WaterOne.

The attached WaterOne Borrow Area Typical Cross Section and table summarize the borrow material needs for Argentine, Armourdale and CID-KS. See "WaterOne Borrow Area Typical Cross Section", for a typical cross section for the borrow area excavation.

Fifty acres will be needed for Armourdale's 350,000 bank cubic yard requirement. This assumes that impervious material is obtained in a 3-foot layer with random material obtained below that from a 2 to 3-foot layer. For all of borrow requirements for Argentine, Armourdale, and CID-KS, approximately 100 acres will be needed assuming impervious material is obtained from a 3-foot layer and random obtained from a 2 to 3 foot layer below that. The impervious layer thickness assumption is critical to estimating the acreages to be used from WaterOne. The 3-foot impervious thickness assumption was derived from the eight boring logs.

Cultural resource investigation into the WaterOne borrow site resulted in a maximum excavation depth of 10 feet. The US Army Corps of Engineers recommended a maximum excavation depth of 10 feet and the Kansas State Historic Preservation Officer has concurred with Corps' recommendation that no archeological survey is needed for borrow activity kept to a depth of 10 feet or less.

BORROW AREA SEARCH

Originally, the Argentine & Armourdale foreshore areas were considered due to their close proximity to the Argentine unit. As HTRW investigations were undertaken for areas of interest, however, various regions of contamination were discovered which eliminated most of these areas from consideration. Total remaining available fill in these areas, ASSUMING NO FURTHER HTRW DISCOVERIES, is approximately 143,000 CY (see FIGURE 1 - "FORESHORE"). Figure 1 reflects avoidance of known HTRW concerns, a minimum 300' standoff distance from existing levees or floodwalls, and maximum depth of excavation of ordinary high water (OHW) minus 4 feet. It is recommended that this area be retained for further consideration during project engineering & design, though there is a possibility that further HTRW investigations will make even the remaining material unusable. Even if no further HTRW issues are discovered, any borrow from this area would need chemical analysis sampling at a rate of 1 sample (about $1000) per 5000 cy of borrow due to the known contamination and associated legal entanglements in the area.

Since the remaining foreshore quantity alone (assuming future HTRW clearance) is marginal for the N500 raise and insufficient for the other two prospective modifications to the Argentine unit, efforts were taken to identify alternative borrow areas as close to the project as possible. FIGURE 2 - "VICINITY MAP" shows various sites considered and investigated. Many of the sites near the project area were either very small or had other undesirable characteristics such as extremely high land values or prior industrial use. Several areas, as discussed on the following pages, were further investigated.

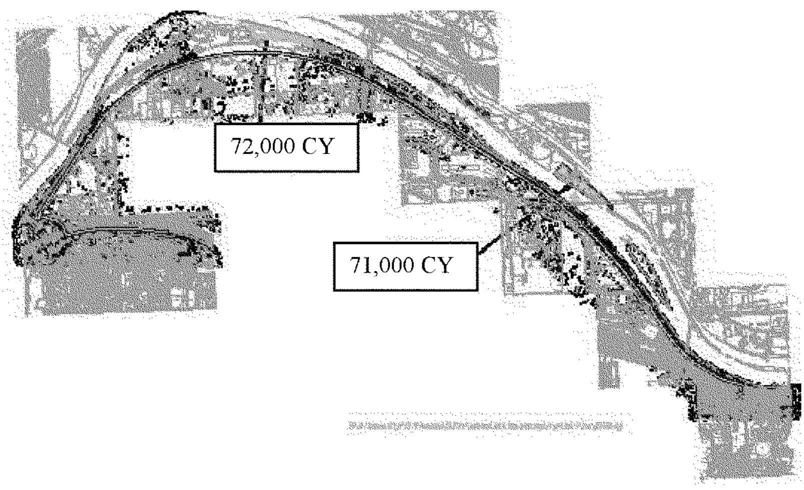


FIGURE 1 – FORESHORE

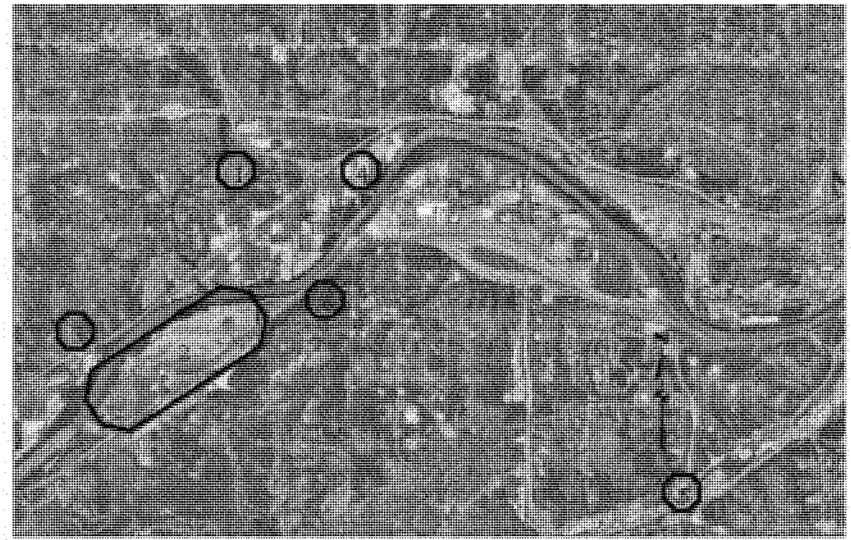


FIGURE - 2 VICINITY MAP

Area 1 is an open field south of the Turner Diagonal. The area is approximately 40 acres and appears to have been previously used as a borrow area. Since access to and from the area requires travel through residential neighborhoods on narrow routes, the area was not considered for further study.

Area 2 is in the Kansas River floodplain. The area is approximately 500 acres, 380 of which are owned by Water District One of Johnson County (WaterOne). This area appeared to be a good candidate for further consideration, and is discussed in detail below.

Area 3 is owned by Amino Brothers Construction and has previously been used as a source of borrows. Approximately 200,000 cy of material is available, per conversation with the owner. This site may be a viable backup source for impervious materials, if required.

Area 4 is approximately 50 acres and used for a variety of commercial / industrial purposes. Since current appraised land values are in excess of $2000 per acre, this area was not considered for further study.

Area 5 is owned by Sandifer Leasing and has previously been used as a source of borrows. Field investigations show little to no remaining fill, therefore the site was not considered for further study.

Area 6 is a large wooded hillside, which appears to be undisturbed. The area below is covered by a network of tunnels, originally used for limestone mining and currently for cold storage. Due to the likelihood of disturbing the tunnels below during earth moving operations, this area was not considered for further study.

See TABLE 1 "BORROW AREA COMPARISON" for a summary comparison of prospective sites.

TABLE 1 - BORROW AREA COMPARISON

AREA	OWNER	HAUL DIST	PROS	CONS	ACTION
1	Unknown	2 miles	Close to site	Residential access, small	Remove from consideration
2	WaterOne	4 miles	Little or no cost	Haul distance	Investigate as primary source
3	Amino Bros.	5 miles	Bank source – expected to be impervious	Haul distance, cost of fill	Keep for possible contingency
4	5701 LLC	2 miles	Close to site	High cost of comm/ind property, developed	Remove from consideration
5	Sandifer	5 miles	None	Haul distance, look like no fill left	Remove from consideration
6	Unkonwn	3 miles	Bank source- expected impervious	Haul distance, likelihood of damaging tunnels below	Remove from consideration
Foreshore	KVDD easement	0 (Argentine) 4 miles (Armourdale)	Very close to site	Potential HTRW, legal entanglements, high chemical sampling cost	Keep for possible contingency

Area 2, shown below in additional detail in FIGURE 3, contains approximately 500 acres and is bounded by the Kansas River and Holliday Drive. WaterOne owns 380 acres in this area and uses the site for disposal of quicklime used in the water treatment process. Individual cells, each 5-10 acres and 20 feet deep, are excavated and, over the course of 3-5 years, filled with dewatered lime (40-60% solids). The cells are then capped with soil, and the excess soil stockpiled elsewhere onsite. During an October 2004 meeting with WaterOne staff, the requirements for the Argentine levee raise project were discussed in detail. WaterOne staff indicated a desire to dispose of excess materials and was interested in pursuing an agreement for use of the excess materials. Soil boring logs for previous WaterOne well and disposal cell construction indicate significant deposits of silt and silty clay, both of which would qualify as impervious fill, in the area.

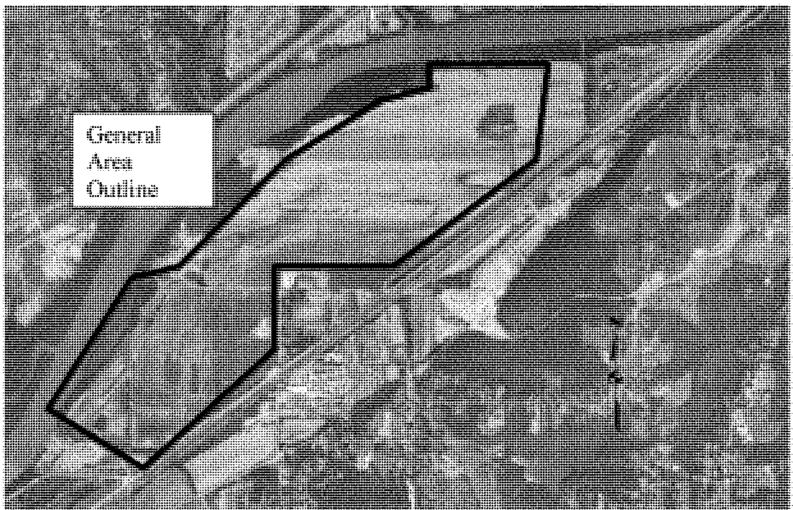


FIGURE 3 – BORROW AREA 2 - WATER ONE

Exploratory soil borings and chemical analysis sampling was conducted in January 2005. Chemical analysis entailed 3 grab samples for volatile organic compounds (VOCs) and three composite samples for metals, pesticides herbicides, and semivolatile organic compounds (SVOCs). Chemical analysis sampling points differed from soil boring locations, but were taken at various locations throughout the WaterOne property to assure representative results. All parameters tested were below action levels.

Subsurface Investigation.

Exploratory Borings.

The subsurface investigation of the borrow area consisted of 8 exploratory borings, 10 - feet deep, drilled with 3 ¾ ID Hollow Stem Auger with 3-inch inner barrel sampler. The borings location with WaterOne property delineated is shown in FIGURE 4 and the strip logs are included at the end of the paragraph. All holes were backfilled prior to leaving the site. No water was encountered during drilling or 24 hours after drilling. Forty (40) jar samples and 8 sack samples (1 composite sack sample for each boring) were collected from all borings. The boring logs show an impervious soil layer consisting of silts and clays extending up to 6 feet below the surface followed by a sandy aquifer. The central part of the borrow area has a thin layer of sand at the surface, varying between 1 and 4.5 feet in thickness, followed by 3 to 4 feet of silts and clay, on the top of the sandy aquifer. The sandy material can be used as backfill in the random portion of the landside levee embankment.

Laboratory Testing.

Selected samples of material obtained during the field exploration were tested to determine engineering and physical properties of the soils. Laboratory testing was performed by Geotechnology, Inc. The laboratory testing included Atterberg Limits, natural moisture contents, and Standard Proctor tests. The samples were grouped in 5 categories of similar characteristics and Atterberg Limits were performed on a representative sample of each category. The moisture content varies between 4 and 35%. Overburden clay and silt material was classified in accordance with ASTM D 2487 as lean clay (CL) or silt (ML). Three of the groups were determined to be non plastic, the other 2 groups were classified one as a lean clay (CL) and the other as silt (ML). The silt was determined to be non-plastic material. The Liquid Limit (LL) of the CL material varies between 39 and 47 and the Plasticity Index (PI) between 19 and 28. The results of the natural moisture content tests and performed on twenty five (25) disturbed samples and of the Atterberg Limits tests performed on 2 selected representative samples of clay material are shown in an enclosure at the end of the paragraph.

Three Standard Proctor Tests were performed on composite samples collected from the borrow areas conform ASTM D-698. The materials were classified as low plasticity clay with the LL between 52 and 55 and PI between 35 and 37 respectively. The maximum dry density varied between 107.5 and 102 pcf with the optimum moisture content varying between 18.5% and 20.5%.

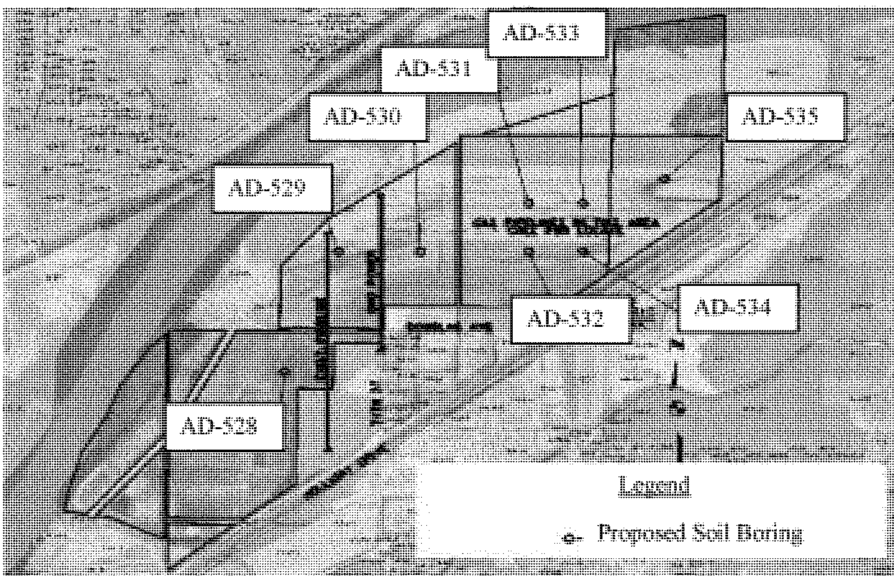
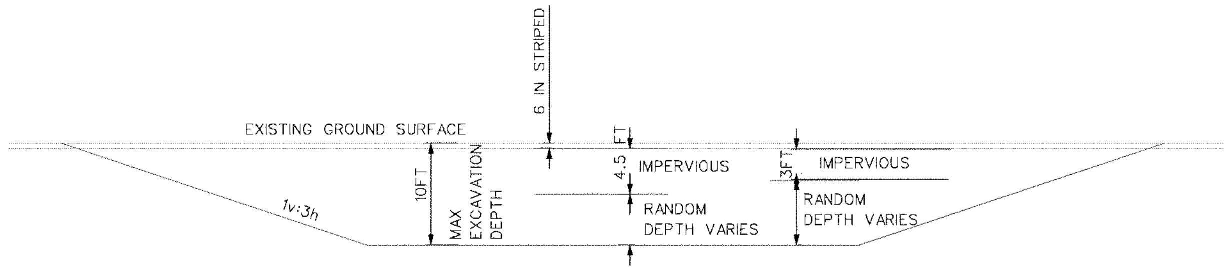


FIGURE 4 - SOIL BORINGS

U.S. Army Corps of Engineers Kaw Valley Drainage District 0680806.52KM
Argentine and Armourdale Levee Unit – Borrow Area
Feasibility Study

KANSAS CITYS LEVEES
FEASIBILITY PHASE 2

WATERONE BORROW AREA
TYPICAL CROSS SECTION



Borrow Area Needs - Current Recommendation N500+3
Using Two Depths (3' and 5') for Impervious Excavation

					Impervious		Random - Alt. 1 [2]		Random - Alt. 2 [3]		Total		
	Total Borrow Quantity	Impervious	Random	Strip Top 6"	Depth of Excavation	Area of Excavation	Depth of Excavation	Area of Excavation	Depth of Excavation	Area of Excavation	Maximum Depth of Excavation [1]	Minimum Depth of Excavation [4]	Total Area of Excavation [5]
	BCY	BCY	BCY		Feet	Acres	Feet	Acres	Feet	Acres	Feet	Feet	Acres
Armourdale	440,500	279,558	160,895	0.5	3	58	3	33	2	58	10	5	58
	440,500	279,558	160,895	0.5	5.0	35	4.5	22	3	35	10	8	35
CID-KS	79,700	59,331	20,371	0.5	3	12	3	4	1	12	10	4	12
	79,700	59,331	20,371	0.5	5.0	7	4.5	3	2	7	10	7	7
Argentine	282,700	144,226	138,490	0.5	3	30	3	29	3	30	10	6	30
	282,700	144,226	138,490	0.5	5.0	18	4.5	19	4.5	19	10	10	19
Total	802,900	483,115	319,756	0.5	3	100	3	66	2	100	10	5	100
	802,900	483,115	319,756	0.5	5.0	60	4.5	44	3	60	10	8	60

Notes

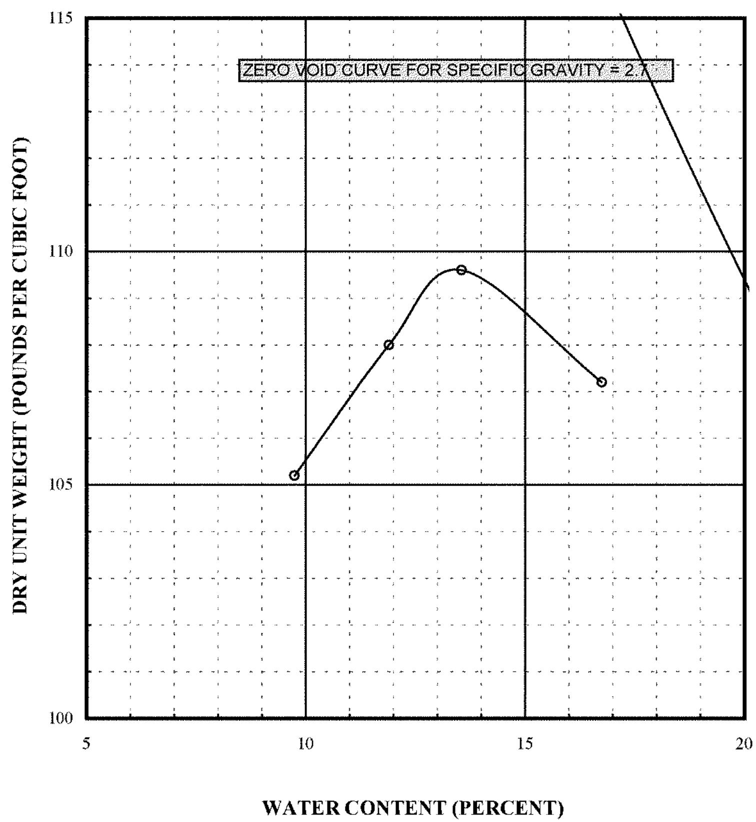
CCY Compacted Cubic Yards
BCY Bank Cubic Yards = CCY/0.8
BOTEA Back of the Envelope Analysis

Assumptions: Impervious material is generally located above other material
Estimate will be revised as levee/floodwall alignment is refined.
(1) Maximum depth of excavation is set to 10 feet.
Kansas State Historic Preservation Officer has concurred with Corps' recommendation that no archeological survey is needed for borrow activity kept to a depth of 10' or less
(2) Alternative 1 is when random excavation depth is equal to impervious excavation depth
(3) Alternative 2 is when random excavation area is equal to impervious excavation area (this minimizes depth of total excavation)
(4) Minimum excavation depth is calculated using Alternative 2 - setting the acreage needed for random fill equal to that needed for impervious fill.
(5) Total area of excavation is the greater value returned when comparing the Impervious Area of Excavation and the Random Alt. 1 Area of Excavation

By: Melissa Corkill
Date: December 14, 2006

	BORING NO.	Sample Depth (feet)	Sample No.	Group Class.	LABORATORY TESTS: Moisture Content %	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Standard Proctor: Max. Dry Density	Standard Proctor: Optima Water Content	Classification or Group Classification ASTM D2487
					ASTM D2216	ASTM D4318			ASTM 698 Method A		
SACK SAMPLES	AD-528	0.8-4.0	Sack-1	--	--	29	21	8	104.7	18.2	CL-dark brown sandy low plasticity CLAY
	AD-529	3.0-6.0	Sack-1	--	--	27	17	10	113.9	14.6	SC-dark brown clayey SAND
	AD-535	1.0-4.5	Sack-1	--	--	47	19	28	102.5	19.0	CL-dark brown sandy low plasticity CLAY
DISTURBED SAMPLES	AD-528	0.0-0.5	J-1	5	29.2	Not enough sample					**Group Classification Number:**
		0.5-0.8	J-2	5							1. CL - dark brown sandy low plasticity CLAY
		0.8-4.0	J-3	1							2. ML - light brown low plasticity SILT
		4.0-4.6	J-4	1	25.0	39	20	19			3. SP - tan fine-grained SAND
		4.6-8.4	J-5	3							4. SM - dark brown silty SAND
		8.4-9.0	J-6	2							5. FILL - dark brown gravelly low plasticity CLAY
	AD-529	0.0-1.0	J-1	4	27.5						with sand
		1.0-3.0	J-2	3							6. CL - dark brown low plasticity CLAY
		3.0-4.0	J-3	4	21.2						
		4.5-6.0	J-4	1	24.9						
		6.0-10.0	J-5	3							
	AD-530	0.0-1.0	J-1	4							
		1.0-1.5	J-2	3	4.0						
		1.5-2.5	J-3	2	20.1	Non-plastic					
		2.5-4.3	J-4	2	11.3						
		4.3-8.0	J-5	2	25.2						
		8.0-9.3	J-6	4		Non-plastic					

BORING NO.	Sample Depth (feet)	Sample No.	LABORATORY TESTS: Group Class.	Moisture Content %	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Standard Proctor: Max. Dry Density	Optima Water Content	Classification or Group Classification ASTM D2487
				ASTM D2216	ASTM D4318			ASTM 698 Method A		
AD-531	0.0-1.0	J-1	4	14.4						
	1.0-3.5	J-2	1	22.2						
	3.5-4.0	J-3	2							
	4.0-6.5	J-4	2	25.1						
	6.5-9.0	J-5	3		Non-plastic					
AD-532	0.0-4.3	J-1	1	26.3						
	4.3-6.3	J-2	1	30.7						
	6.3-8.3	J-3	2							
AD-533	0.0-2.3	J-1	1	18.9						
	2.3-4.3	J-2	2	9.9						
	4.3-6.0	J-3	2	14.9						
	6.0-8.0	J-4	1	26.2						
	8.0-9.3	J-5	2							
ADU-534	0.0-4.0	J-1	1	25.7						
	4.0-6.0	J-2	2	30.7						
	6.0-6.5	J-3	6	35.4						
	6.5-7.0	J-4	1	28.3						
	7.0-9.0	J-5	2	15.5						
AD-535	0.0-4.5	J-1	6	20.3	47	19	28			
	4.5-7.5	J-2	1	29.5						
	7.5-8.5	J-3	2							
	8.5-9.5	J-4	3							
	9.5-10.0	J-5	2							



PROJECT NAME

Argentine Levee Unit – Borrow Area

SPECIFICATIONS

Standard Proctor
ASTM D 698 Method A
Percent of Compaction N/A
Moisture Range ±% N/A

PROCTOR TEST RESULTS

Max. Dry Density	Optimum Water Content
104.7 pcf	18.2%

ATTERBERG LIMITS (ASTM D-4318)

Liquid Limit	Plastic Limit	Plasticity Index
29	21	8

DESCRIPTION

Dark brown, sandy low plasticity CLAY

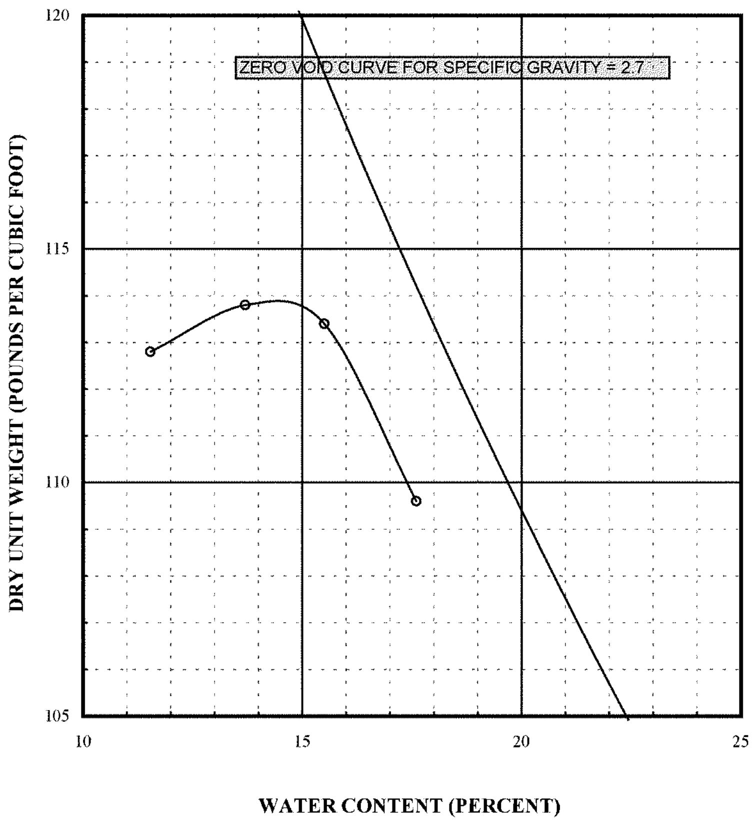
SAMPLE LOCATION

AD 528, 0.8-4.0 feet below grade

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
St. Louis, Collinsville, Kansas City

COMPACTION TEST

Job No.	0680806.52KM	**Test Date**	3/4/2005
Sampled By	USCOE	**Tested By**	SD
Sample Date	1/18/2005	**Calc. By**	YAW
Proctor No.	1075	**Ch'd By**	ARK



PROJECT NAME

Argentine Levee Unit – Borrow Area

SPECIFICATIONS

Standard Proctor
ASTM D 698 Method A
Percent of Compaction N/A
Moisture Range ±% N/A

PROCTOR TEST RESULTS

Max. Dry Density	Optimum Water Content
113.9 pcf	14.6%

ATTERBERG LIMITS (ASTM D-4318)

Liquid Limit	Plastic Limit	Plasticity Index
27	17	10

DESCRIPTION

Dark brown clayey SAND

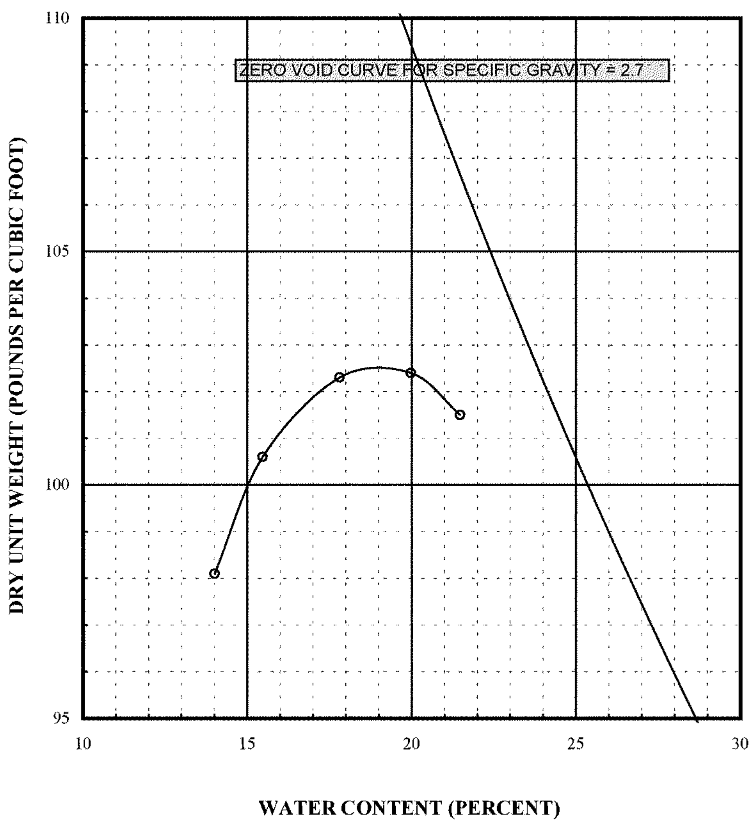
SAMPLE LOCATION

AD 529, 3.0-6.0 feet below grade

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
St. Louis, Collinsville, Kansas City

COMPACTION TEST

Job No.	0681901.32KT	**Test Date**	3/7/2005
Sampled By	USCOE	**Tested By**	SD
Sample Date	1/18/2005	**Calc. By**	YAW
Proctor No.	1077	**Ch'd By**	ARK



PROJECT NAME

Argentine Levee Unit – Borrow Area

SPECIFICATIONS

Standard Proctor
ASTM D 698 Method A
Percent of Compaction N/A
Moisture Range ±% N/A

PROCTOR TEST RESULTS

Max. Dry Density	Optimum Water Content
102.5 pcf	19.0%

ATTERBERG LIMITS (ASTM D-4318)

Liquid Limit	Plastic Limit	Plasticity Index
47	19	28

DESCRIPTION

Dark brown sandy low plasticity CLAY

SAMPLE LOCATION

AD 535, , 1.0-4.5 feet below grade

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
St. Louis, Collinsville, Kansas City

COMPACTION TEST

Job No.	0681901.32KT	**Test Date**	3/4/2005
Sampled By	USCOE	**Tested By**	SD
Sample Date	1/18/2005	**Calc. By**	YAW
Proctor No.	1076	**Ch'd By**	ARK

LOG OF BORING AD-528

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-528
LOCATION: Kansas and Missouri
COORDINATES: N 14190735.79, E 1141877.29 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

FIELD DATA: DEPTH (ft) | SOIL SYMBOL | BREAKS: bb or mb | SAMPLE/DRILL METHOD | BLOWS | T: TORVANE KG/CM SQ | RC: % RQD: % Additional Field Data

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney Geologist:Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

LABORATORY DATA

OTHER LAB DATA: S: Minus 200 Sieve (%) U: Unconfined Compressive Strength (tsf) C: Confining Pressure (psi) F: Failure Strain (%) T: Total Sulfates P: Soil pH

Depth (ft)	DESCRIPTION OF STRATUM	USCS SYMBOL	LL	PI	MOISTURE CONTENT (%)	VG=Visual Grouping FC=Field Classification	OTHER LAB DATA
0–0.5	CLAY HARD DRY BROWN frozen (0.5)				29.2	VG5	
0.5–0.8	GRAVELLY COBBLES MODERATELY HARD GREY decomposed rock (0.8)					VG5	
0.8–4.6	LEAN CLAY MEDIUM DAMP - MOIST DARK BROWN (4.6)	CL	29	8		VG1	
		CL	39	19	25	VG1	
4.6–8.4	FINE SAND LOOSE - MEDIUM COMPACT DRY LIGHT BROWN (8.4)					VG3	
8.4–10.0	SILT LOOSE DRY GREY (10.0)					VG2	

Bottom of hole - No Refusal
Backfilled to surface with cuttings and 3 bags Holeplug

LEGEND: Fill (made ground); USCS Low Plasticity Clay; USCS Poorly-graded Sand; USCS Silt

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML; VG3 - SP; VG5 - FILL

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-529

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-529
LOCATION: Kansas and Missouri
COORDINATES: N 14192413.68, E 1142543.89 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

FIELD DATA

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney Geologist: Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

LABORATORY DATA

OTHER LAB DATA: S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH

DEPTH (ft)	DESCRIPTION OF STRATUM	USCS SYMBOL	LL	PI	MOISTURE CONTENT (%)	VG=Visual Grouping / FC=Field Classification	OTHER LAB DATA
0–1.0	FINE SAND, FROZEN, DARK BROWN				27.5	VG4	
1.0–3.0	FINE SAND, LOOSE, DRY, BROWN					VG3	
3.0–4.5	CLAYEY SAND, MEDIUM COMPACT, DAMP-MOIST, DARK BROWN	SC	27	10	21.2	VG4	
4.5–6.0	CLAY, SOFT, DAMP, DARK BROWN, very silty				24.9	VG1	
6.0–10.0	FINE SAND, LOOSE-MEDIUM, DRY-DAMP, LIGHT BROWN, silty					VG3	

Bottom of hole - No Refusal
Backfilled to surface with cuttings and 3 bags Holeplug

LEGEND: USCS Silty Sand; USCS Poorly-graded Sand; USCS Clayey Sand; USCS Low Plasticity Clay

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG3 - SP; VG4 - SM

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-530

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-530
LOCATION: Kansas and Missouri
COORDINATES: N 14192416.14, E 1143534.5 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/12/05 - 1/18/05

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney   Geologist: Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling   ▼ Water level after drilling

FIELD DATA columns: DEPTH (ft); SOIL SYMBOL; BREAKS: bb or mb; SAMPLE/DRILL METHOD; BLOWS; T: TORVANE KG/CM SQ; RC: %, RQD: %, Additional Field Data

LABORATORY DATA columns: USCS SYMBOL; ATTERBERG LIMITS: LIQUID LIMIT (LL), PLASTIC INDEX (PI); MOISTURE CONTENT (%); VG=Visual Grouping, FC=Field Classification; OTHER LAB DATA (S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH)

Depth (ft)	Description of Stratum	Bottom depth	LL	PI	Moisture Content (%)	VG/FC	Other Lab Data
0	SILTY SAND, FROZEN, DARK BROWN, fine grained	1.0				VG4	
1.0		1.5			4	VG3	
1.5	FINE SAND, LOOSE, DRY-DAMP, BROWN, poorly graded	2.5			20	VG2	
2.5	SILT, MEDIUM COMPACT, DAMP, DARK BROWN	4.3			11	VG2	
4.3	SILT, MEDIUM COMPACT, LIGHT BROWN				25	VG2	
	SILT, MEDIUM COMPACT, DAMP, GRAYISH BROWN, sandy; wet zone	8.0					
8.0	SILTY SAND, MEDIUM COMPACT, DAMP, LIGHT BROWN, laminated, fine grained	10.0				VG4	

Bottom of hole - No Refusal
Backfilled to surface with cuttings and 3 bags Holeplug

LEGEND: USCS Silty Sand; USCS Poorly-graded Sand; USCS Silt

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG2 - ML; VG3 - SP; VG4 - SM

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-531

SHEET 1 of 1

US Army Corps of Engineers ®
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-531
LOCATION: Kansas and Missouri
COORDINATES: N 14193052.59, E 1144847.77 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

FIELD DATA

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney Geologist: Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

LABORATORY DATA

Field data columns: DEPTH (ft); SOIL SYMBOL; BREAKS: bb or mb; SAMPLE/DRILL METHOD; BLOWS; T: TORVANE KG/CM SQ; RC: %, RQD: %, Additional Field Data

Other lab data: S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH

DEPTH (ft)	DESCRIPTION OF STRATUM	USCS SYMBOL	LL	PI	MOISTURE CONTENT (%)	VG=Visual Grouping FC=Field Classification	OTHER LAB DATA
0–1.0	SILTY SAND, FROZEN, LIGHT BROWN, very fine grained				14	VG4	
1.0–3.5	LEAN CLAY, SOFT-VERY SOFT, DAMP, DARK BROWN, very silty ~ 30-40 % silt				22	VG1	
3.5–4.0	SILT					VG2	
4.0–6.5	LOOSE, DRY, BROWN, with fine sand; SILT, MEDIUM COMPACT, DAMP, BROWN, slightly sandy ~ 10-15 % very fine sand				25	VG2	
6.5–10.0	FINE SAND, MEDIUM COMPACT - LOOSE, DAMP-DRY, LIGHT BROWN					VG3	

Bottom of hole - No Refusal
Backfilled to surface with cuttings and 3 bags Holeplug

LEGEND

- USCS Silty Sand
- USCS Low Plasticity Clay
- USCS Silt
- USCS Poorly-graded Sand

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML; VG3 - SP; VG4 - SM

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-532

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-532
LOCATION: Kansas and Missouri
COORDINATES: N 14192422.33, E 1144971.11 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

FIELD DATA

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney Geologist: Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

LABORATORY DATA

OTHER LAB DATA: S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH

DEPTH (ft)	SOIL SYMBOL	BREAKS: bb or mb	SAMPLE/DRILL METHOD	BLOWS	T: TORVANE KG/CM SQ	RC: % RQD: % Additional Field Data	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL	LIQUID LIMIT LL	PLASTIC INDEX PI	MOISTURE CONTENT (%)	VG=Visual Grouping FC=Field Classification	OTHER LAB DATA
0							LEAN CLAY MEDIUM MOIST DARK BROWN frozen to 1.0 ft 4.3					26	VG1	
4.3							LEAN CLAY MEDIUM MOIST-WET DARK BROWN silty 6.3					31	VG1	
6.3							SILT MEDIUM COMPACT DRY-DAMP LIGHT BROWN sandy 10.0	USCS Low Plasticity Clay USCS Silt					VG2	
10							Bottom of hole - No Refusal Backfilled to surface with cuttings and 3 bags Holeplug							

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-533

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-533
LOCATION: Kansas and Missouri
COORDINATES: N 14193066.09, E 1145518.24 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney Geologist:Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

FIELD DATA columns: DEPTH (ft); SOIL SYMBOL; BREAKS: bb or mb; SAMPLE/DRILL METHOD; BLOWS; T: TORVANE KG/CM SQ; RC: %, RQD: %, Additional Field Data

LABORATORY DATA columns: USCS SYMBOL; ATTERBERG LIMITS (LIQUID LIMIT LL, PLASTIC INDEX PI); MOISTURE CONTENT (%); VG=Visual Grouping, FC=Field Classification; OTHER LAB DATA (S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH)

Depth (ft)	DESCRIPTION OF STRATUM	Bottom depth	USCS SYMBOL	LL	PI	MOISTURE CONTENT (%)	VG/FC	OTHER LAB DATA
0	LEAN CLAY, MEDIUM, DAMP, DARK BROWN, silty	2.3				19	VG1	
2.3	SILT, MEDIUM COMPACT, DRY-DAMP, LIGHT BROWN, with fine-grained sand	4.3				10	VG2	
4.3	SILT, MEDIUM COMPACT, DRY-DAMP, BROWN, with very fine-grained sand	6.0				15	VG2	
6.0	LEAN CLAY, MEDIUM, MOIST, DARK BROWN, with silt	8.0				26	VG1	
8.0	SILT, MEDIUM COMPACT, DRY-DAMP, LIGHT BROWN, with very fine-grained sand	10.0					VG2	

Bottom of hole - No Refusal
Backfilled to surface with cuttings and 3 bags Holeplug

LEGEND: USCS Low Plasticity Clay; USCS Silt

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-534

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-534
LOCATION: Kansas and Missouri
COORDINATES: N 14192405.24, E 1145517.16 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/19/05 - 1/19/05

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driler: Mike Cooney   Geologist:Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling   ▼ Water level after drilling

FIELD DATA: DEPTH (ft); SOIL SYMBOL; BREAKS: bb or mb; SAMPLE/DRILL METHOD; BLOWS; T: TORVANE KG/CM SQ; RC: %, RQD: %, Additional Field Data

LABORATORY DATA: USCS SYMBOL; ATTERBERG LIMITS: LIQUID LIMIT (LL), PLASTIC INDEX (PI); MOISTURE CONTENT (%); VG=Visual Grouping, FC=Field Classification; OTHER LAB DATA (S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH)

Depth (ft)	DESCRIPTION OF STRATUM	Bottom depth	USCS SYMBOL	LL	PI	MOISTURE CONTENT (%)	VG/FC	OTHER LAB DATA
0	LEAN CLAY, MEDIUM, DAMP, DARK BROWN, silty	4.0				26	VG1	
4	SILT, MEDIUM COMPACT, WET, DARK BROWN, clayey	6.0				31	VG2	
6	LEAN CLAY, SOFT, MOIST-WET, DARK BROWN, silty	6.5				35	VG6	
6.5	LEAN CLAY, MEDIUM, MOIST-WET, DARK BROWN, silty	7.0				28	VG1	
7	SILT, MEDIUM COMPACT, DRY-DAMP, LIGHT BROWN, with very fine-grained sand	10.0				16	VG2	

Bottom of hole - No Refusal
Backfilled to surface with cuttings and 3 bags Holeplug

LEGEND
USCS Low Plasticity Clay
USCS Silt

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML; VG6 - CL(LL=47,PI=28)

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-535

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-535
LOCATION: Kansas and Missouri
COORDINATES: N 14193402.93, E 1146504.01 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

FIELD DATA — DEPTH (ft), SOIL SYMBOL, BREAKS: bb or mb, SAMPLE/DRILL METHOD, BLOWS, T: TORVANE KG/CM SQ, RC: % RQD: %, Additional Field Data

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driler: Mike Cooney Geologist:Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

LABORATORY DATA — USCS SYMBOL; ATTERBERG LIMITS: LIQUID LIMIT (LL), PLASTIC INDEX (PI); MOISTURE CONTENT (%); VG=Visual Grouping FC=Field Classification; OTHER LAB DATA (S: Minus 200 Sieve (%), U: Unconfined Compressive Strength (tsf), C: Confining Pressure (psi), F: Failure Strain (%), T: Total Sulfates, P: Soil pH)

DEPTH (ft)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL	LL	PI	MOISTURE CONTENT (%)	VG/FC	OTHER LAB DATA
0–4.5	LEAN CLAY SOFT DAMP DARK BROWN silty ~ 10-15% silt		CL CL	47 47	28 28	20	VG6	
4.5–7.5	LEAN CLAY MEDIUM WET DARK BROWN silty					30	VG1	
7.5–8.5	SILT MEDIUM COMPACT DAMP LIGHT BROWN with very fine-grained sand	USCS Low Plasticity Clay					VG2	
8.5–9.5	FINE SAND LOOSE DRY LIGHT BROWN poorly graded	USCS Silt					VG3	
9.5–10.0	SILT MEDIUM COMPACT DAMP-MOIST BROWN sandy	USCS Poorly-graded Sand					VG2	

Bottom of hole - No Refusal
Backfilled to surface with cuttings and 3 bags Holeplug

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML; VG3 - SP; VG6 - CL(LL=47,PI=28)

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

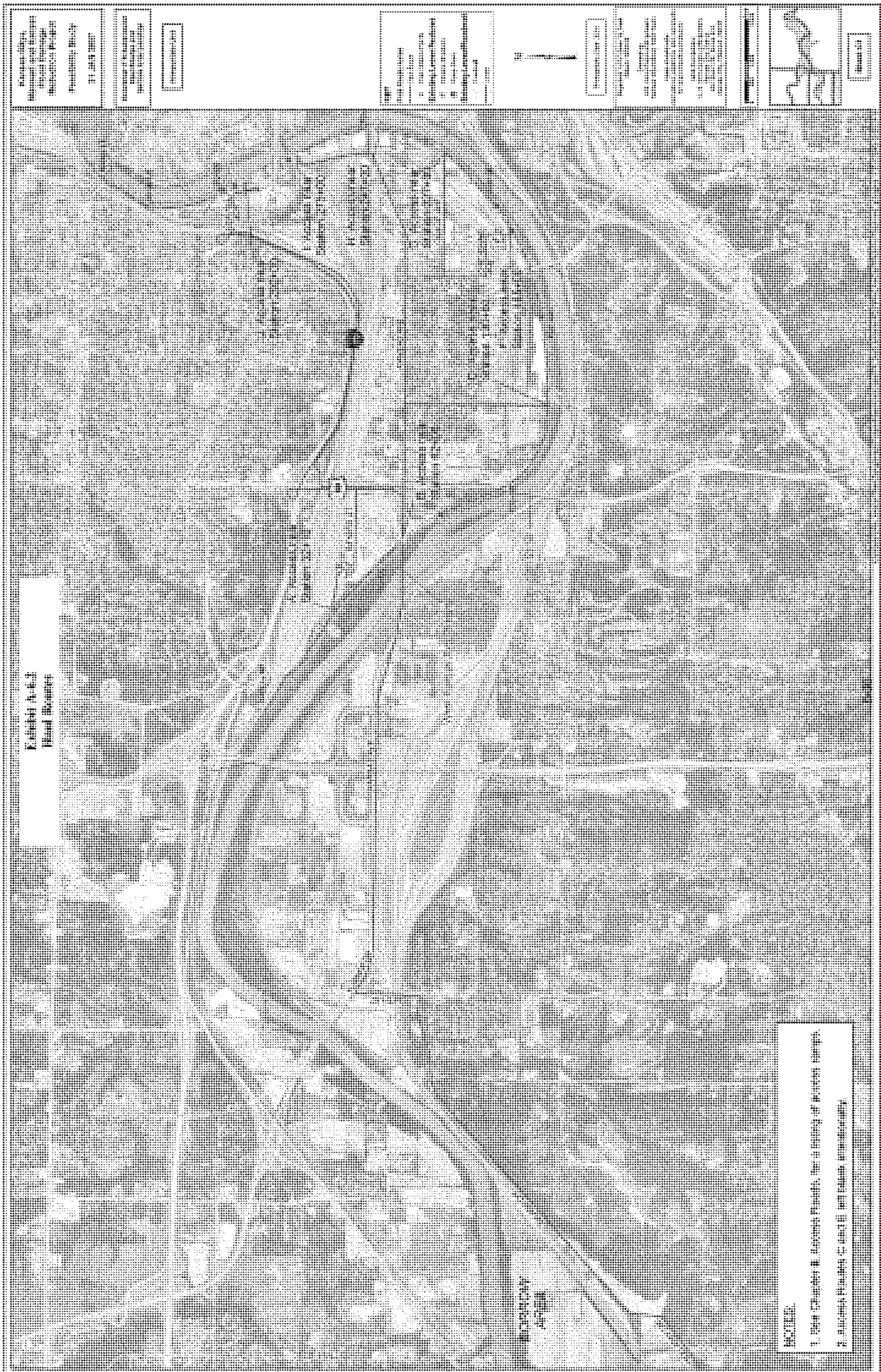


EXHIBIT A-6.3

Kansas City's Levee and Floodwall Gravity and Utility Pipeline Guidance

KANSAS CITY'S LEVEE AND FLOODWALL GRAVITY AND UTILITY PIPELINE GUIDANCE

PURPOSE

The purpose of this document is to provide specific guidance during the feasibility phase of the Kansas City's Levee project as to the disposition of existing utilities and drainage structures within the sections of levee and floodwall to be raised. This guidance will be used for the feasibility level of effort in order to develop reasonable costs associated with the modification of drainage structures and the relocation of utilities.

Uplift of utilities within the critical zone of the levee or floodwall will be addressed in accordance with COE criteria. Uplift is not addressed in this KCL guidance.

REFERENCES

	Local Protection – Web page guidance
	Local Protection - Guidebook on web page (Guidance for work Proposed Near or within Federally Constructed Flood Risk Management Projects)
EM 1110-2-1913	Design and Construction of Levees
EM 1110-2-2902	Conduits, Culverts, and Pipes
EM 1110-2-3102	General Principles of Pumping Station Design and Layout
EM 1110-2-3104	Structural and Architectural Design of Pumping Stations
EM 1110-2-3105	Mechanical and Electrical Design of Pumping Stations (Changes 1 of 2)

GRAVITY PIPELINES

Existing pipelines crossing the levee that do not meet current COE criteria shall be replaced with pipelines that are compliant. Existing pipelines that meet current COE criteria shall remain with the following exceptions:

Any Corrugated Metal Pipe (CMP) with a diameter greater than 36" shall be replaced with a minimum diameter 48" Reinforced Concrete Pipe (RCP).

Any pipe inadequate to handle the drainage shall be replaced with a minimum diameter 48" RCP.

Any pipe known to have joints that are not watertight shall be replaced with a minimum diameter 48" RCP.

For new pipe installations, CMP will not be allowed.

Pipe strengths, unless otherwise known, will be assumed to be that required by Corps criteria at the time of their installation. Pipe condition shall be determined by field assessment.

GATEWELLS AND POSITIVE CLOSURES

In areas where levee raises are performed, positive closure will be provided for all drainage and utility lines crossing the levee. EM 1110-2-1913 states that gravity lines that penetrate the embankment or foundation of a levee must be provided with devices to assure positive closure. This criteria also states that gravity lines should be provided with flap-type or slide-type service gates on the riverside of the levee. Because the KS River and MO River are not fast rising rivers, a flap gate will not be recommended on existing outfalls where sluice gates are present but no flap gate. For new outfall structures, however, flap gates will generally be installed.

Emergency means of closure is suggested for gravity lines in addition to the positive closure device. Historically, a flap gate on the end of the pipe has acted as this second closure device. However, it is possible to use sandbags or concrete to fill a gatewell as a means of emergency closure during a flood situation, although this is not the recommended alternative.

All gatewells within the Kansas City Levee study area are considered confined spaces. OSHA regulations and Corp EM 385-1-1 require anyone entering a confined space to comply with specific confined space entry requirements. New or modified gatewells will be designed so that these confined space entry requirements can be met. For example, space will be provided above the gatewell opening so that a tripod can be set to facilitate non-entry rescue.

NON-GRAVITY PIPELINES CROSSING THROUGH OR UNDER LEVEES

It is preferable for all pipes or conduits to cross over the levee rather than penetrate the embankment or foundation materials. This includes pipes carrying fiber optic, pressurized gas or pressurized liquid. Where raises are made to the levee, existing non-gravity pipelines should be relocated over the crest of the new levee raise. See detail "Typical Utility Crossing Levee Raise". A determination to relocate existing lines will be made on a case-by-case basis.

Pressure pipe

All pipes allowed to penetrate the embankment or foundation of a levee must be provided with devices to assure positive closure. These valves shall be placed in close proximity to the levee and have capability to be closed rapidly to prevent gas or fluid from escaping within or beneath a levee should the pipe rupture within these areas.

Provisions for closure of pressure pipes on the water side must also be provided to prevent backflow of floodwater into the protected area should the pipe rupture.

Casing Pipes, Cased Pipes and Conduits Crossing Through or Under Levees (Telecommunications)

It is preferred that conduits or casing pipes cross up and over the levee. However, where it is not possible to go over the levee, casing pipes or conduits must be installed in accordance with COE criteria. Refer to COE Guidebook located on the KC District website for directional drilling procedures.

ABANDONED PIPELINES

Pipelines, which are currently abandoned and grouted in accordance with COE criteria under or through the levee, will not be disturbed. Pipes that have been abandoned and do not meet criteria or it is unknown if they meet criteria shall be removed or properly abandoned according to COE criteria. Pipelines that are currently active but are to be abandoned as part of this project will be removed or abandoned according to COE criteria.

Removal

For feasibility purposes only, the following guidance is used in determining if an abandoned pipeline will be removed or abandoned in-place in accordance with Corps criteria.

> Where levee heights are less than 10 feet and when an abandoned utility is buried less than 5 feet below the base of the levee, the abandoned utility crossing under the levee should be removed unless special circumstances warrant a different approach.

Exploration Trench

For cost estimating purposes during feasibility, all known pipes are assumed to be located as shown on maps and plans or as located in the field during feasibility site visits.

No exploration trenches will be specified during feasibility. However, it is noted that during PED phase, it may be determined that exploration trenches will be needed during construction in order to find some utilities or to verify that some utilities do not exist as shown on the drawings.

Grouting Abandoned Pipelines

If a pipe does not meet the requirement for removal, the pipe should be properly abandoned by filling with a grout based substance, e.g., cement-bentonite, or flowable fill. The grout or flowable fill mix should be approved by the Corps of Engineers. The grout shall be fluid enough, and pumped in the up-slope direction so that the pipe will be completely filled leaving no voids. Points of access need to be made into the pipe at sufficient intervals to accomplish the grouting, see detail "Typical Utility Abandonment" for additional details regarding abandoning a utility in place.

OTHER CONSIDERATIONS

Special consideration will be given to existing pipe crossings on a case-by-case basis when hazardous, toxic, and radioactive waste (HTRW) concerns or real estate issues exist. HTRW concerns exist in various locations along the Kansas City Seven Levee system. When it is not desirable to disturb the existing ground due to HTRW concerns, the final recommendation for removing/relocating an existing utility will weigh the risks involved with disturbing the ground against leaving an existing utility in place. When real estate issues exist, the final recommendation will consider how real estate is affected.

SUMMARY OF RECOMMENDATIONS

For sections of levee or floodwall to be raised or modified, current Corps requirements will be extended to all components of that levee section, including any pipes and closure structures therein. When it is not practical to meet Corps requirement, each utility will be evaluated on a case-by-case basis.

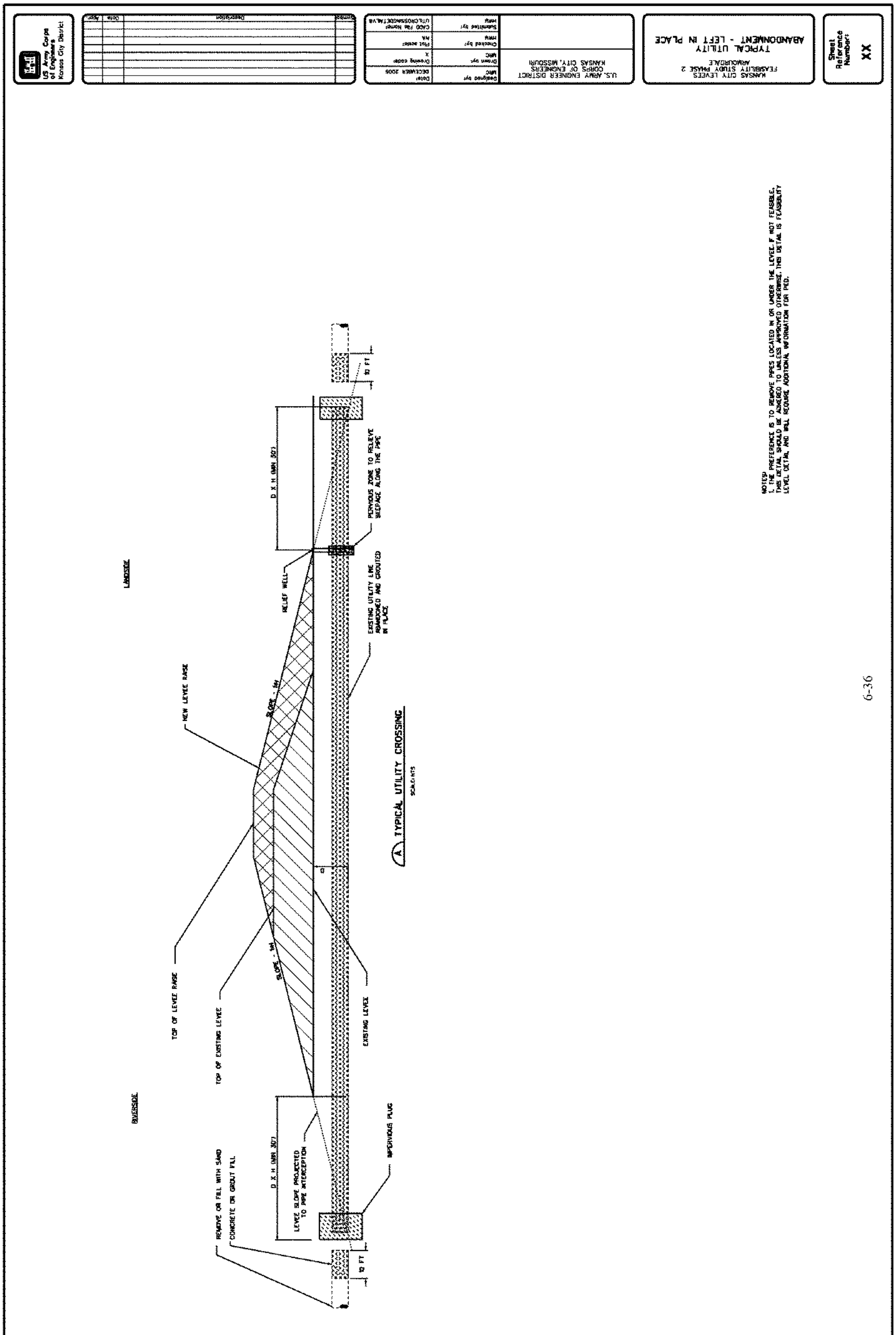

RIVERSIDE
LANDSIDE
TOP OF LEVEE RAISE
NEW LEVEE RAISE
REMOVE OR FILL WITH SAND
CONCRETE OR GROUT FILL
TOP OF EXISTING LEVEE
D X H (MIN 30')
SLOPE - 1H
LEVEE SLOPE PROJECTED TO PIPE INTERCEPTION
RELIEF WELL
D
10 FT
EXISTING LEVEE
EXISTING UTILITY LINE ABANDONED AND GROUTED IN PLACE
PERVIOUS ZONE TO RELIEVE SEEPAGE ALONG THE PIPE
IMPERVIOUS PLUG
A TYPICAL UTILITY CROSSING
SCALE: NTS
NOTES:
1. THE PREFERENCE IS TO REMOVE PIPES LOCATED IN OR UNDER THE LEVEE. IF NOT FEASIBLE, THIS DETAIL SHOULD BE ADHERED TO UNLESS APPROVED OTHERWISE. THIS DETAIL IS FEASIBILITY LEVEL DETAIL AND WILL REQUIRE ADDITIONAL INFORMATION FOR PED.
US Army Corps of Engineers Kansas City District
Symbol
Description
Date
Appr.
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI
Designed by: MRC
Drawn by: MRC
Checked by: HMM
Submitted by: HMM
Date: DECEMBER 2006
Drawing code: X
Plot scale: NA
CADD file Name: UTILCROSSINGDETAIL.V8
KANSAS CITY LEVEES
FEASIBILITY STUDY PHASE 2
ARMOURDALE
TYPICAL UTILITY
ABANDONMENT - LEFT IN PLACE
Sheet Reference Number:
XX

EXHIBIT A-6.4

Armourdale Utility Crossings: Inventory and Action for N500+3 Raise

Armourdale Utility Crossings: Inventory and Action for N500+3 Raise
Created by: Melissa Corkill
Peer Reviewed by: Hank Mildenberger

Date: 23-Aug-2006
Modified: 02-Mar-2007

UTILITY IDENTIFICATION			UTILITY CROSSING GENERAL INFORMATION AND RECOMMENDED ACTION FOR N500+3 RAISE								
Station (ft.)	Conduit Size	Conduit Function	Flow Direction	Flow Type	Conduit Composition	Conduit Length Below Flood Protection (ft.) Existing Conditions	Control Structure Type	Depth below existing levee crest (ft)	Owner/ Compensable	Comments	Action
-9+63 (0+75 UE)	36"	City: San. Sewer Force Main	Riv to Land	Pressure	DIP	Under Floodwall	Valve	17' / Invert Elev. = 757		Structure is part of RR closure gap. Forcemain is buried about 2.5' to 3' below grade outside of levee/floodwall	Relocate forcemain around the stop log gap/high ground tie-in. Gatewell structure and valve should be located at highpoint of line to allow positive closure of pipeline. Remove the abandoned part of the forcemain between existing stop log structure and riverside of new stop log structure. Fill gatewell and pipe within gatewell with flowable fill.
-4+58 (5+80 UE)	*	Fiber Optic	NA	NA	*	200	None - NA	0.5"		2 cables	Relocate fiber optic lines up and over levee. Remove abandoned fiber optic.
7+50 UE	24"	Storm Sewer	Land to Riv	Gravity	RCP	100	Sluice and Flap	Invert Elev 755.9			No Action. SEE STRUCTURAL EVALUATION
12+79 UE	12'x8'	Storm Sewer	Land to Riv	Gravity	RCB	150	Sluice	25' below levee		Drains Matoon Creek, Riverside Gatewell Structure	No Action. SEE STRUCTURAL EVALUATION
4+93 (15+33 UE)	8'x8'	Storm Sewer	Land to Riv	Gravity	*	150	Sluice and Flap	Elev ~740			No Action. SEE STRUCTURAL EVALUATION
28+22	6"	Sand Pipe	NA	*	SP	30	None	~4.5' / Elev 769		American Sand Company -- Pipe Abandoned	Find and Remove
28+30	18"	Sand Pipe	NA	*	SP	30	None	~ 5' / Elev 768.5		American Sand Company -- Pipe Abandoned	Find and Remove
28+33	6"	Sand Pipe	NA	*	SP	30	None	~ 4.5' / Elev 769		American Sand Company -- Pipe Abandoned	Find and Remove
32+80	42"	Storm Sewer	Land to Riv	Gravity	RCP	80	Sluice and Flap	33' / Invert Elev 750		Installed 1980	SEE STRUCTURAL EVALUATION
41+45	48"	San. Sewer Force Main	NA	Pressure	CIP	240	Valve	30' / Invert Elev 740		Removed	No Action
52+45	3"	Water for Proctor and Gamble Fire Testing Area	Land to Riv	Pressure	DIP	120	Post Indicator Valve	4' below levee crest		Pipe is uncovered along portions of the riverside levee.	Relocate up and over floodwall. Water line is not used every day or every week. Remove abandoned line once line is relocated.
58+44	36"	Storm Sewer	Land to Riv	Gravity	RCP w/ Steel Liner	140	Gate Valve and Flap	20' - 27'		Abandoned. Once used to drain cooling water from Proctor & Gamble	Find and Grout
61+81	36"	Storm Sewer	Land to Riv	Gravity	CIP	200	Gate Valve and Flap	23'		Abandoned. Proctor & Gamble used to drain cooling water	Find and Grout

Armourdale Utility Crossings: Inventory and Action for N500+3 Raise
Created by: Melissa Corkill
Peer Reviewed by: Hank Mildenberger

Date: 23-Aug-2006
Modified: 02-Mar-2007

UTILITY IDENTIFICATION			UTILITY CROSSING GENERAL INFORMATION AND RECOMMENDED ACTION FOR N500+3 RAISE								
62+10	18"	Sanitary Sewer	NA	Gravity	SP	50	*	16'		Abandoned	Remove (as long as floodwall is replaced with levee)
62+10	24"	Gas	NA	Pressure	SP	Above Floodwall	NA	Above Floodwall		On Kansas Avenue Bridge	No Action
62+10	24"	Gas	NA	Pressure	SP	Under Floodwall	Gate	15' / Elev 758.1		Abandoned. Located under former stop log gap used prior to KS Ave. Bridge Installation. No info on removal but KGE replaced with line on bridge.	Remove (as long as floodwall is replaced with levee)
62+65	12"	Water	NA	Pressure	SP	Above Floodwall	NA	Above Floodwall		On Kansas Avenue Bridge	No Action
62+65	10"	Water	NA	Pressure	SP	Under Floodwall	*	Elev 758.7		Abandoned	Remove (as long as floodwall is replaced with levee)
62+70	6"	Fiber Optic	NA	Electrical	Duct	Above Floodwall	NA	1" Above Floodwall		Near Kansas Avenue Bridge	Relocate over floodwall raise
64+71	42"	Storm Sewer	Land to Riv	Gravity	RCP	Under Floodwall	Sluice and Flap	30' / Invert Elev 742		Drainage Structure adjacent to floodwall	No Action. SEE STRUCTURAL EVALUATION
75+12	48"	KAW PP Recirculation Line	*	Gravity	RCP	160 (floodwall)	Power Plant Process Controls Flow of Water through Pipe	15' to 20'/Centerline Elev 751.7		KAW Power Plant	Remove 48" line from sealing weir to landside toe of levee and continue removal under levee/floodwall to the intake structure. Do not re-establish utility.
75+22	6"	KAW PP Intake Water	*	Pressure	SP	160 (floodwall)	*	18' below floodwall top		KAW Power Plant	Remove pipe. Do not re-establish utility.
75+32	72"	KAW PP Intake Influent #1	Riv to Land	Pressure	RCP	160 (floodwall)	Power Plant Intake Pumps Control Flow of Water through pipe	25' to 30' below floodwall top		KAW Power Plant	Remove pipe. Do not re-establish utility.
75+45	3"	KAW PP Intake Chlorine	*	*	SP or PVC	160 (floodwall)	*	8' to 11' below floodwall top		KAW Power Plant	Remove pipe. Do not re-establish utility.
75+50	1" OR 2"	KAW PP Intake Gas	Land to Riv	Pressure	SP	160 (floodwall)	*	18" below floodwall top		KAW Power Plant	Remove pipe. Do not re-establish utility.
75+50	4"	KAW PP Conduit	NA	Electrical	PVC	160 (floodwall)	NA	8' to 11' below floodwall top		KAW Power Plant	Remove conduit. Do not re-establish utility.
75+62	17"W X 43"H	KAW Power Plant Electrical	NA	Electrical	DUCT	160 (floodwall)	NA	8' below floodwall top		KAW Power Plant - Duct has 12-4" inner ducts	Remove duct bank. Do not re-establish utility
75+89	72"	KAW PP Intake Influent #2	Riv to Land	Pressure	RCP	160 (floodwall)	Power Plant Intake Pumps Control Flow of Water through Pipe	27' to 32' below floodwall top		KAW Power Plant	Remove pipe. Do not re-establish utility.

Armourdale Utility Crossings: Inventory and Action for N500+3 Raise
Created by: Melissa Corkill
Peer Reviewed by: Hank Mildenberger

Date: 23-Aug-2006
Modified: 02-Mar-2007

UTILITY IDENTIFICATION			UTILITY CROSSING GENERAL INFORMATION AND RECOMMENDED ACTION FOR N500+3 RAISE									
75+90	2"	Water Emergency	Land to Riv	Pressure	SP	160 (floodwall)	Valve	4' to 9' below floodwall top		Normally not under pressure	Remove pipe. Do not re-establish utility.
76+63	72"	Osage Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL.	RCP	NA	Sluice and Drop Leaf	30+ below levee			SEE STRUCTURAL EVALUATION AND PUMP PLANT EVALUATION - Reconnect discharge pipes through cutoff wall
79+60	84"	KAW PP Discharge	Land to Riv	Gravity	RCP	180	Sluice Gate	40+ below levee			Grout from seiling weir to drainage structure. Remove from drainage structure to outfall.
79+80	60"	KAW PP Discharge	Land to Riv	Gravity	RCP	180	Sluice Gate	40+ below levee			Grout from seiling weir to drainage structure. Remove from drainage structure to outfall.
81+07	2.5" in 4"	Calcium Carbonate	NA	*	SP in SP	150	*	20' below levee		Abandoned. Discharged to KAW PP discharge headwall area	Find and Grout
90+79	30"	Sanitary Sewer (inverted)	Riv to Land	Gravity	DIP	125	Sluice Gate	40' below levee	City of KCK		No Action. SEE STRUCTURAL EVALUATION
91+76	30"	Storm Sewer	Land to Riv	Gravity	CIP	125	Sluice and Flap	40' below levee			No Action. SEE STRUCTURAL EVALUATION
99+63	4"	Discharge Line	Land to Riv	Gravity	SP	150	*	6' below levee	Private	Could not located outfall or inlet source. Likely abandoned	Find and Remove
108+95	42"	Sanitary Sewer	Riv to Land	Gravity	RCP	NA	Sluice Gate (Riverside)	40'+ below levee	City of KCK	Argentine Inter. River Crossing, 42" through levee, 30" under River	No Action. SEE STRUCTURAL EVALUATION
127+20	24"	Potable Water	NA	Pressure	CIP	120	Gate Valve	22' below levee	BPU	Abandoned but no appearance of being grouted	Grout line and valve pit. Fill pipe on riverside of levee with sand.
129+20	5'X6'	12th St. Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL.	RCB	120	Sluice Gate	Gravity: 30' to 40' below levee			No Action. SEE STRUCTURAL EVALUATION
129+20	20" x 2	12th St. Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL.	DIP	120	NA	Elev 767.2 (~3')		These two lines are the pump station pressurized discharge lines that are routed over the levee into the gatewell	Relocate pressure discharge pipe up and over levee. And remove abandoned lines from pump station to gatewell.
129+60	72"	12th St. Storm Sewer	Land to Riv	Gravity	BRICK	60	*	*		Plugged and filled with impervioius or pervious mateiral. Brick sewer for 12th Street PS area. Not Active.	No Action
156+75	6'X8'	Mill St. Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL.	RCB	120	Sluice Gate	30' to 40' below levee		Riverside gatewell	No Action. SEE STRUCTURAL EVALUATION
156+75	20" x 2	Mill St. Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL.	DIP	120	NA			These two lines are the pump station pressurized discharge lines that are routed over the levee into the gatewell	Relocate pressure discharge pipe up and over levee. And remove abandoned lines from pump station to gatewell.
172+05	12"	KS Gas	NA	Pressure	APCS	NA	NA	NA		Routed over levee on 7th Street bridge	No Action

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UTILITY IDENTIFICATION			UTILITY CROSSING GENERAL INFORMATION AND RECOMMENDED ACTION FOR N500+3 RAISE								
185+70	72"	5th St. Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL.	RCP	120	Sluice and Flap	40' below levee	City of KCK		No Action. SEE PUMP PLANT EVALUATION
186+74	30"	Sanitary Sewer	Riv to Land	Gravity	DIP	100	Sluice Gate	44' below ground		Greystone Heights Sewer River Crossing	No Action. SEE STRUCTURAL EVALUATION
194+60	18" and 12"	Midwest Cold Storage Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL.	CIP and CIP	100	Sluice and Flap	18" invert ~740 12" invert ~746		Pump Plant abandoned. These two lines parallel each other, one 6' above the other and connect the landside and riverside levee.	Grout in both pipes but during PED confirm if the 12" line is still connected and in use by Midwest Cold Storage.
212+76	24"	Storm Sewer	Land to Riv	Gravity	CIP	120	Sluice and Flap	30' below levee			No Action. SEE STRUCTURAL EVALUATION
220+64	24	Storm Sewer	Land to Riv	Gravity	CIP	120	Sluice and Flap	30'+ below levee			No Action. SEE STRUCTURAL EVALUATION
230+77	7.5' X 7.5'	Shawnee Ave Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL.	RCB	110	2 Sluice and 2 Flap	35' below levee			No Action. SEE PUMP PLANT EVALUATION
231+38.91	57"	Storm Sewer	Land to Riv	Gravity	Concrete	100	None	30' below levee		Plugged at both ends but could not find.	Find and Grout
240+73	48"	Storm Sewer	Land to Riv	Gravity	RCP	110	Sluice and Flap	30'+ below levee			No Action. SEE STRUCTURAL EVALUATION
244+70	24"	Storm Sewer	Land to Riv	Gravity	CIP	90	Sluice and Flap	25' below levee			No. Action. SEE STRUCTURAL EVALUATION
246+53	30"	Storm Sewer Forcemain	Land to Riv	Pressure	CIP	100	Valve and Flap	30' below levee		Termination of Railroad Storm Force Main	No Action. Pipe may freeze if routed over floodwall. If levee is placed here then should consider routing up and over. SEE STRUCTURAL EVALUATION
250+31	12"	Storm Sewer	Land to Riv	Gravity	CIP	118	Gate Valve and Flap	25' below floodwall top			No Action
253+43	12"	Storm Sewer	Land to Riv	Gravity	CIP	118	Gate Valve and Flap	25' below levee		Abandoned and Grouted	No Action
256+71	12"	Storm Sewer	Land to Riv	Gravity	CIP	100	Gate Valve and Flap	25' below levee		Abandoned	Grout Pipe and Fill Gatewell
260+00	30"	Storm Sewer	Land to Riv	Gravity	CIP	100	Gate Valve and Flap	30'+ below floodwall top			No Action. SEE STRUCTURAL EVALUATION
260+84	12"	Storm Sewer	Land to Riv	Gravity	CIP	40	Gate Valve and Flap	25' below floodwall top		Abandoned	Grout Pipe and Fill Gatewell
262+69	12"	Storm Sewer	Land to Riv	Gravity	CIP	50	Gate Valve and Flap	25' below floodwall top			No Action. SEE STRUCTURAL EVALUATION
266+76	16"	Storm Sewer	Land to Riv	Gravity	CIP	70	Sluice and Flap	25' below floodwall top			No Action. SEE STRUCTURAL EVALUATION
276+00	NA	Fiber Optic	NA	NA	NA	NA	NA	NA			Relocate over floodwall raise
276+79	42"	Kansas City Southern Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL.	CIP	80	Sluice and Flap	35' below floodwall top		Slot Area	No Action. SEE STRUCTURAL EVALUATION AND PUMP STATION EVALUATION.
281+50	42"	Storm Sewer	Land to Riv	Gravity	RCP	110	Sluice and Flap	35' below floodwall top			No Action. SEE STRUCTURAL EVALUATION
286+59	24"	PBI Gordon Pump Plant Gravity Storm Flow	Land to Riv	SEE PUMP STATION EVAL.	CIP	110	Sluice and Flap	30'+ below floodwall top		Pipe under floodwall through piles	No Action. SEE STRUCTURAL EVALUATION

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UTILITY IDENTIFICATION			UTILITY CROSSING GENERAL INFORMATION AND RECOMMENDED ACTION FOR N500+3 RAISE								
286+59	6"	PBI Gordon Pump Plant Discharge Pipe	Land to Riv	SEE PUMP STATION EVAL.	*					6" discharge through floodwall.	SEE STRUCTURAL EVALUATION AND PUMP PLANT EVALUATION. Relocate 6" discharge up and over floodwall. Fill in abandoned pipe through floodwall.
290+52	24"	Storm Sewer	Land to Riv	Gravity	CIP	110	Sluice and Flap	30'+ below floodwall top		Located under piles	No Action. SEE STRUCTURAL EVALUATION
295+45	18"	Storm Sewer	Land to Riv	Gravity	RCP	*	Sluice Gate	25' below floodwall top		Landside Gatewell is at grade located west of pump station	No Action. SEE STRUCTURAL EVALUATION
295+52	24"	National Beef Pumping Plant / KCK Pump Plant / (Central Ave. PP - KVDD)	Land to Riv	SEE PUMP STATION EVAL.	CIP	90	Sluice and Flap	30' below floodwall top	City of KCK	10" pump discharge penetrates floodwall	PENDING PUMP STATION EVALUATION
295+79	2"	National Beef Pumping Plant / KCK Pump Plant / (Central Ave. PP - KVDD)	Land to Riv	Pressure	SP	Through floodwall	Valve	8' below floodwall top		Plug at both ends. Exposed on riverside of floodwall. Buried on landside of floodwall	PENDING PUMP STATION EVALUATION
295+80	10"	National Beef Pumping Plant / KCK Pump Plant / (Central Ave. PP - KVDD)	Land to Riv	*	CIP	Through floodwall	Flap and Valve	8' below floodwall top		Exposed on riverside of floodwall. Buried on landside of floodwall	PENDING PUMP STATION EVALUATION
299+20	3"	Central Ave. Pumping Plant - Sump Pump	Land to Riv	SEE PUMP STATION EVAL.	CIP		Gate Valve	10' below floodwall top		Discharge is through wall	PENDING PUMP STATION EVALUATION
299+20	2-14"	Central Ave. Pump Plant - Discharge	Land to Riv	SEE PUMP STATION EVAL.	CIP		2 gate valves and 2 flap gates	10' below floodwall top		Discharge is through wall	PENDING PUMP STATION EVALUATION
311+11	7.5' X 7.5'	Storm Sewer	Land to Riv	Gravity	RCB	90	Sluice Gate	33	City of KCK		No Action. SEE STRUCTURAL EVALUATION
315+10	5' X 4'	Storm Sewer	Land to Riv	Gravity	RCB	80	Sluice Gate	25			No Action. SEE STRUCTURAL EVALUATION
324+58 (41+45 LE)	48"	Sanitary Force Main	Land to Riv	Pressure	CIP	180	Gate Valve (Riverside)	18	City of KCK	Armourdale Interceptor	No Action. SEE STRUCTURAL EVALUATION
328+18 (45+05 LE)	42"	Storm Sewer/Sanitary Sewer	Land to Riv	Gravity	CMP	60	Unknown	16		Existing conditions: Drains high ground. With levee raise, will need new pipe and closure structure. This is also a CSO.	No Action: No longer part of protected area
329+35 (46+22 L.E.)	24"	Storm Sewer	Land to Riv	Gravity	CMP	50	Flap Gate	5		Existing conditions: Drains high ground. With levee raise, will need new pipe and closure structure	No Action: No longer part of protected area
50+00	8"	KS Gas	NA	Pressure	APCS	NA	NA	NA		Routed over levee on bridge	No Action
334+08 (50+95 LE)	24"	Storm Sewer	Land to Riv	Gravity	CMP	100	Flap Gate	5			No Action: No longer part of protected area
334+83 (51+70 L.E.)	12"	Spring Sump Drain	Land to Riv	Gravity	*	40	NA	NA		Spring Sump Drain - Located on Riverside of Levee and does not cross levee	No Action

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339+54 (56+41 LE)	24"	Storm Sewer	Land to Riv	Gravity	CMP	100	Flap Gate	5		Existing conditions: Drains high ground. With levee raise, will need new pipe and closure structure	No Action: No longer part of protected area
339+63 (56+60 LE)	1 - 36"	Sanitary Force Main	Land to Riv	Pressure	DIP	NA	1 Gate Valve	NA		River crossing: Armourdale Forcemain and Fairfax Forcemain. Armourdale crossed unit at 41.45 LE	No Action
340+49 (57+36 LE)	12"	Storm Sewer	Land to Riv	Gravity	CMP	50	Flap Gate	4		Possibly Plugged. Historical conditions: Drained high ground. With levee raise, will need closure structure if still in use	No Action: No longer part of protected area
341+43 (58+30 LE)	25" X 16"	Storm Sewer	Land to Riv	Gravity	CMP Arch	100	NA	6		Plugged	No Action: No longer part of protected area
343+59 (60+45 LE)	25" X 16"	Storm Sewer	Land to Riv	Gravity	CMP Arch	75	Flap Gate	5		Existing conditions: Drains high ground. With levee raise, will need new pipe and closure structure	No Action: No longer part of protected area
LE		Oxygen		Pressure					Praxair	Oxygen line	No Action: No longer part of protected area

Table Legend and Notes

*	Information not found
APCS	Anode Protected Coated Steel
CIP	cast iron pipe
CMP	corrugated metal pipe
DIP	ductile iron pipe
DS	Drainage Structure
FW	Floodwall
Land to Riv	Landside to Riverside
LEV	Levee
NA	Not Applicable
OH	Overhead utility (does not penetrate flood protection)
PP	Pump Plant
RCB	Reinforced Concrete Box
RCP	Reinforced Concrete Pipe
Riv to Land	Riverside to Landside
SL	Stop Log Gap
SP	steel pipe

All Drainage Structures that say No Action, mean there is no action recommended for utility outlet relocation.
Structural or pumping analysis may indicate other recommendations and should be referred to for gatewell structural analysis

Direction of Flow for sanitary sewers was obtained from Wyandotte County Sewer AutoCad files

All elevations presented in this spreadsheet are based on NGVD 29

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UTILITY IDENTIFICATION			CASE BY CASE REVIEW MEETING WITH LOCAL PROTECTION SECTION: NOTES FOR EACH UTILITY CROSSING
Station (ft.)	Conduit Size	Conduit Function	Case-by-Case Review Conducted Feb. 20, 2007. See meeting minutes for general recommendations. Where Feb 26 meeting over rides Feb 20 recommendations, it is so noted at the end of the comment.
-9+63 (0+75 UE)	36"	City: San. Sewer Force Main	FEB 20: Reroute forcemain around the stop log gap/high ground tie-in. This would essentially be up and over line of protection. Gatewell structure and valve should be located at the highpoint of line to allow positive closure of pipeline as well as provide a siphon breaker. Will need to remove the abandoned part of the forcemain between existing stop log structure and riverside of new stop log structure. Fill gatewell and pipe within gatewell with flowable fill up to the ground surface.
-4+58 (5+80 UE)	*	Fiber Optic	FEB 20: Reroute fiber optic lines up and over levee. Remove abandoned fiber optic
7+50 UE	24"	Storm Sewer	FEB 20: OK
12+79 UE	12'x8'	Storm Sewer	FEB 20: It is OK that there is no flap gate on this discharge line. The primary closure device is the sluice gate, the emergency closure can be to fill the gatewell with sand.
4+93 (15+33 UE)	8'x8'	Storm Sewer	FEB 20: OK
28+22	6"	Sand Pipe	FEB 20: During construction, investigate on the riverside of the levee by digging a trench to 1 foot below the recorded depth of the pipes and 10' either side of the recorded station of the pipe. Remove piping believed to be about 50' long. Backfill trench. If pipe is not encountered at expected depth, contractor shall consult with COE.
28+30	18"	Sand Pipe	FEB 20: During construction, investigate on the riverside of the levee by digging a trench to 1 foot below the recorded depth of the pipes and 10' either side of the recorded station of the pipe. Remove piping believed to be about 50' long. Backfill trench. If pipe is not encountered at expected depth, contractor shall consult with COE.
28+33	6"	Sand Pipe	FEB 20: During construction, investigate on the riverside of the levee by digging a trench to 1 foot below the recorded depth of the pipes and 10' either side of the recorded station of the pipe. Remove piping believed to be about 50' long. Backfill trench. If pipe is not encountered at expected depth, contractor shall consult with COE.
32+80	42"	Storm Sewer	FEB 20: OK
41+45	48"	San. Sewer Force Main	FEB 20: OK
52+45	3"	Water for Proctor and Gamble Fire Testing Area	FEB 20: Once line is relocated, remove old line from its riverside end point to the landside tie-in point.
58+44	36"	Storm Sewer	FEB 20: Because this line is deep, grout in the pipe beneath the footprint of the levee, construct an impervious cut-off area at the toe of the levee on the riverside, install a vertical drain on the landside toe of the levee if the bedding material warrants or can't be inspected. FEB 26: See meeting minutes and detail for additional information
61+81	36"	Storm Sewer	FEB 20: Because this line is deep, grout in the pipe beneath the footprint of the levee, construct an impervious cut-off area at the toe of the levee on the riverside, install a vertical drain on the landside toe of the levee if the bedding material warrants or can't be inspected. FEB 26: See meeting minutes and detail for additional information

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UTILITY IDENTIFICATION			CASE BY CASE REVIEW MEETING WITH LOCAL PROTECTION SECTION: NOTES FOR EACH UTILITY CROSSING
62+10	18"	Sanitary Sewer	FEB 20: Because this line is deep, grout in the pipe beneath the footprint of the levee, construct an impervious cut-off area at the toe of the levee on the riverside, install a vertical drain on the landside toe of the levee if the bedding material warrants or can't be inspected, FEB 26: Remove pipeline since this floodwall is being replaced with a levee and the line is not that deep below the footing of the floodwall.
62+10	24"	Gas	OK
62+10	24"	Gas	FEB 20: Because this line is deep, grout in the pipe beneath the footprint of the levee, construct an impervious cut-off area at the toe of the levee on the riverside, install a vertical drain on the landside toe of the levee if the bedding material warrants or can't be inspected, FEB 26: Remove pipeline since this floodwall is being replaced with a levee and the line is not that deep below the footing of the floodwall.
62+65	12"	Water	OK
62+65	10"	Water	FEB 20: Because this line is deep, grout in the pipe beneath the footprint of the levee, construct an impervious cut-off area at the toe of the levee on the riverside, install a vertical drain on the landside toe of the levee if the bedding material warrants or can't be inspected, FEB 26: Remove pipeline since this floodwall is being replaced with a levee and the line is not that deep below the footing of the floodwall.
62+70	6"	Fiber Optic	OK
64+71	42"	Storm Sewer	OK
75+12	48"	KAW PP Recirculation Line	FEB 20: On the landside, remove the 48" line from the sealing weir to the landside toe of the levee. (about 440' of removal). Grout in the line under the floodwall up to within 20' of the intake structure with excavatable flowable fill so that when the slurry wall is installed it cuts through the pipe and the flowable fill. (about 180' of flowable fill) On the riverside, remove the pipe from intake structure back about 20'. FEB 26: Remove 48" line from the sealing weir all the way to the intake structure (640').
75+22	6"	KAW PP Intake Water	FEB 20: Grout in the line under the floodwall with excavatable flowable fill so that when the slurry wall is installed it cuts through the pipe and the flowable fill (about 180' of grout). On the riverside, remove the pipe from the intake structure back about 20'. FEB 26: Remove pipeline since the floodwall is being replaced with a new floodwall and the pipe is not too deep.
75+32	72"	KAW PP Intake Influent #1	FEB 20: Grout in the line under the floodwall up to within 20' of the intake structure with excavatable flowable fill so that when the slurry wall is installed it cuts through the pipe and the flowable fill. (about 180' of flowable fill) On the riverside, remove the pipe from intake structure back about 20'. On the landside, fill with grout or remove within the critical zone. This critical zone removal suggestion will likely not be possible. Should revisit. FEB 26: Remove pipe landside of levee to intake structure.
75+45	3"	KAW PP Intake Chlorine	FEB 20: Remove utility within the footprint of the levee. Will likely not have to remove outside of levee footprint. FEB 26: Remove utility from landside of levee to intake structure. If the pipe also runs along the landside toe, may need to remove.
75+50	1" OR 2"	KAW PP Intake Gas	FEB 20: Remove utility within the footprint of the levee. Will likely not have to remove outside of levee footprint. FEB 26: Remove utility from landside of levee to intake structure. If the pipe also runs along the landside toe, may need to remove.
75+50	4"	KAW PP Conduit	FEB 20: Remove utility within the footprint of the levee. Will likely not have to remove outside of levee footprint.
75+62	17"W X 43"H	KAW Power Plant Electrical	FEB 20: Remove utility within the footprint of the levee. Will likely not have to remove outside of levee footprint.
75+89	72"	KAW PP Intake Influent #2	FEB 20: Grout in the line under the floodwall up to within 20' of the intake structure with excavatable flowable fill so that when the slurry wall is installed it cuts through the pipe and the flowable fill. (about 180' of flowable fill) On the riverside, remove the pipe from intake structure back about 20'. On the landside, fill with grout or remove within the critical zone. This critical zone removal suggestion will likely not be possible. Should revisit. FEB 26: Remove pipe landside of levee to intake structure.

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75+90	2"	Water Emergency	FEB 20: Remove utility within the footprint of the levee. Will likely not have to remove outside of levee footprint.
76+83	72"	Osage Pump Plant Outfall	FEB 20: Need to meet with construction people to determine how reconnection pipe after installation of cutoff wall will be completed.
79+60	84"	KAW PP Discharge	FEB 20: Grout in the line under the floodwall up to within 20' of the intake structure with excavetable flowable fill so that when the slurry wall is installed it cuts through the pipe and the flowable fill. (about 180' of flowable fill) On the riverside, remove the pipe from intake structure back about 20'. On the landside, fill with grout or remove back to sealing weir. FEB 26: Grout the line from the seiling weir to the drainage structure. Remove the pipe from the drainage structure to the outfall structure.
79+80	60"	KAW PP Discharge	FEB 20: Grout in the line under the floodwall up to within 20' of the intake structure with excavatable flowable fill so that when the slurry wall is installed it cuts through the pipe and the flowable fill. (about 180' of flowable fill) On the riverside, remove the pipe from intake structure back about 20'. On the landside, fill with grout or remove back to sealing weir. FEB 26: Grout the line from the seiling weir to the drainage structure. Remove the pipe from the drainage structure to the outfall structure.
81+07	2.5" in 4"	Calcium Carbonate	FEB 20: Grout under the levee. FEB 26: Grout under the levee and remove pipe out to outfall structure.
90+79	30"	Sanitary Sewer (Inverted)	OK
91+76	30"	Storm Sewer	OK
99+63	4"	Discharge Line	FEB 20: During construction, install an inspection trench at the riverside toe to find the pipe, then remove pipe from discharge point back to source which appears to have been an inlet on the private property to the north. (Approximately 200 feet of removal) FEB 26: Remove pipe from 30' from landside toe to outfall Sinde line is shallow.
108+95	42"	Sanitary Sewer	OK
127+20	24"	Potable Water	FEB 20: Remove pipeline from river's edge back to riverside toe of levee (approximately 400'). Although a conservative approach, Local Protection felt that removal of the pipe from the river's edge is necessary to prevent the seepage entry point from moving from the river's edge to the landside end of the abandoned pipe. Fill pipe with grout under the levee footprint. Remove pipeline on the landside within the critical zone. Grout pipe if removal is not possible within the critical zone. Will need to verify that removal within the critical zone is possible under this project. Fill valve pit with grout (box is estimated to be 12' x 12' by 10 feet deep - not field measured.) FEB 26: Grout line under levee and extend grout according to Typical Abandonment Detail
129+20	5'X6'	12th St. Pump Plant Outfall	OK
129+20	20" x 2	12th St. Pump Plant Outfall	FEB 20: Once lines are relocated up and over the levee, remove the abandoned 20" DIPs from Pump Station to Gatewell.
129+60	72"	12th St. Storm Sewer	FEB 20: Since this is a critical sewer, we need to find this sewer during PED and determine exactly where it is and how we will fill it in if possible. FEB 26: Following meeting we found the design memo that stated the following: During 1952 construction, the original brick sewer from the PP to the river "was plugged with concrete at each end, then filled with pervious and impervious material zoned with the restored levee section.", therefore, no action needed.
156+75	6'X8'	Mill St. Pump Plant Outfall	OK
156+75	20" x 2	Mill St. Pump Plant Outfall	FEB 20: Once lines are relocated up and over the levee, remove the abandoned 20" DIPs from Pump Station to Gatewell.
172+05	12"	KS Gas	OK

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UTILITY IDENTIFICATION			CASE BY CASE REVIEW MEETING WITH LOCAL PROTECTION SECTION: NOTES FOR EACH UTILITY CROSSING
185+70	72"	5th St. Pump Plant Outfall	OK
186+74	30"	Sanitary Sewer	OK
194+60	18" and 12"	Midwest Cold Storage Pump Plant Outfall	OK
212+76	24"	Storm Sewer	OK
220+64	24	Storm Sewer	OK
230+77	7.5' X 7.5'	Shawnee Ave Pump Plant Outfall	OK
231+38.91	57"	Storm Sewer	
240+73	48"	Storm Sewer	
244+70	24"	Storm Sewer	
246+53	30"	Storm Sewer Forcemain	
250+31	12"	Storm Sewer	
253+43	12"	Storm Sewer	
256+71	12"	Storm Sewer	
260+00	30"	Storm Sewer	
260+84	12"	Storm Sewer	
262+69	12"	Storm Sewer	
266+76	16"	Storm Sewer	
276+00	NA	Fiber Optic	
276+79	42"	Kansas City Southern Pump Plant Outfall	
281+50	42"	Storm Sewer	
286+59	24"	PBI Gordon Pump Plant Gravity Storm Flow	

Armourdale Utility Crossings: Inventory and Action for N500+3 Raise
Created by: Melissa Corkill
Peer Reviewed by: Hank Mildenberger

Date: 23-Aug-2006
Modified: 02-Mar-2007

UTILITY IDENTIFICATION			CASE BY CASE REVIEW MEETING WITH LOCAL PROTECTION SECTION: NOTES FOR EACH UTILITY CROSSING
286+59	6"	PBI Gordon Pump Plant Discharge Pipe	
290+52	24"	Storm Sewer	
295+45	18"	Storm Sewer	
295+52	24"	National Beef Pumping Plant / KCK Pump Plant / (Central Ave. PP - KVDD)	
295+79	2"	National Beef Pumping Plant / KCK Pump Plant / (Central Ave. PP - KVDD)	
295+80	10"	National Beef Pumping Plant / KCK Pump Plant / (Central Ave. PP - KVDD)	
299+20	3"	Central Ave. Pumping Plant - Sump Pump	
299+20	2-14"	Central Ave. Pump Plant - Discharge	
311+11	7.5' X 7.5'	Storm Sewer	
315+10	5' X 4'	Storm Sewer	
324+58 (41+45 LE)	48"	Sanitary Force Main	FEB 20: Local Protection suggested we leve this pipe in place and create an inpsection program whereby this pipe is taken out of service and inpsected. Local protection is a little concerned that this pipe runs parellel with the levee near the toe for quite a distance (as much as 1000'). FEB 26: Leave pipe in place. See meeting minutes for reasoning.
328+18 (45+05 LE)	42"	Storm Sewer/Sanitary Sewer	
329+35 (46+22 L.E.)	24"	Storm Sewer	
50+00	8"	KS Gas	
334+08 (50+ 95 LE)	24"	Storm Sewer	
334+83 (51+70 L.E.)	12"	Spring Sump Drain	

Armourdale Utility Crossings: Inventory and Action for N500+3 Raise
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Date: 23-Aug-2006
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UTILITY IDENTIFICATION			CASE BY CASE REVIEW MEETING WITH LOCAL PROTECTION SECTION: NOTES FOR EACH UTILITY CROSSING
339+54 (56+41 L.E)	24"	Storm Sewer	
339+63 (56+60 LE)	1 - 36"	Sanitary Force Main	
340+49 (57+36 LE)	12"	Storm Sewer	
341+43 (58+30 LE)	25" X 16"	Storm Sewer	
343+59 (60+45 LE)	25" X 16"	Storm Sewer	
LE		Oxygen	

Armourdale Utility Crossings: Inventory and Action for N500+3 Raise
Created by: Melissa Corkill
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Date: 23-Aug-2006
Modified: 02-Mar-2007

UTILITY IDENTIFICATION			BACKGROUND DOCUMENTS AND RESEARCH NOTES FOR EACH UTILITY CROSSING						
Station (ft.)	Conduit Size	Conduit Function	Reference Sited o&m Manual (1978)	Reference Sited: Operations Manual (1979)	Reference Sited: HNTB	Reference Sited: Site visits in Fall 2006 and winter 2007	Reference Sited: Wyandotte County CAD files (not QC checked by County)	Reference Sited: KVDD Drainage Structure List	Other
-9+63 (0+75 UE)	36"	City: San. Sewer Force Main	Structure is part of RR closure gap. Forcemain is buried about 2.5' to 3' below grade outside of levee/floodwall. Plate 267				Forcemain is buried about 2.5' to 3' below grade outside of levee/floodwall	x	
-4+58 (5+80 UE)	*	Fiber Optic				Conduits are buried less than 2 feet deep			
7+50 UE	24"	Storm Sewer						x	
12+79 UE	12'x8'	Storm Sewer	Could not find pipes during August 2006 site visit.	Elev 746		No flap gate (Jan 2007 site visit)		X Mattoon Creek	
4+93 (15+33 UE)	8'x8'	Storm Sewer	Sluice and Flap					X	
28+22	6"	Sand Pipe	5' below levee	~5' below levee/ Elev 789	Abandoned	Could not find pipes during August 2006 site visit.			
28+30	18"	Sand Pipe	5' below levee	~5' below levee/ Elev 768.5	Abandoned	Could not find pipes during August 2006 site visit.			
28+33	6"	Sand Pipe	5' below levee	~5' below levee/ Elev 769	Abandoned	Could not find pipes during August 2006 site visit.			
32+80	42"	Storm Sewer	Not Shown	33' / Invert Elev 750	Installed 1980			X	
41+45	48"	San. Sewer Force Main		30' / Invert Elev 740. Line is "xxxx" crossed out. Drwg says "valve"			Line not shown. Believed to be removed		
52+45	3"	Water for Proctor and Gamble Fire Testing Area	Not Shown	Not Shown		Pipe is not buried along portions of levee			
58+44	36"	Storm Sewer	20' below levee	27' / Invert Elev 746.5, Gate and Flap	Abandoned			X Not Active, Proctor and Gamble	Design Memo 1971: RCP installed in 1951. 30", 1/2" thick steel liner installed in 1961. Active line as of this 1971 design memo.
61+81	36"	Storm Sewer	23' below levee	23' below levee	Abandoned			X Not Active, Proctor and Gamble	Design Memo 1971: Proctor and Gamble's used to drain cooling water. CIP in good shape.

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Armourdale Utility Crossings: Inventory and Action for N500+3 Raise
Created by: Melissa Corkill
Peer Reviewed by: Hank Mildenberger

Date: 23-Aug-2006
Modified: 02-Mar-2007

UTILITY IDENTIFICATION			BACKGROUND DOCUMENTS AND RESEARCH NOTES FOR EACH UTILITY CROSSING						
62+10	18"	Sanitary Sewer	16' below floodwall	16' below floodwall	Abandoned		There is a 24" VCP MH that starts near the bridge. Possibly started at this MH - MH SAN090-001 (AutoCAD file)		KS DOT Drwgs of KS Ave Bridge say abandoned
62+10	24"	Gas	14' below floodwall						
62+10	24"	Gas	14' below floodwall			KGS does not have record of the buried gas line being removed			
62+65	12"	Water	12' below floodwall						
62+65	10"	Water	12' below floodwall						KS DOT Drwgs of KS Ave Bridge
62+70	6"	Fiber Optic	not shown						
64+71	42"	Storm Sewer		Invert Elev 742				X	KS DOT Drwgs of KS Ave Bridge
75+12	48"	KAW PP Recirculation Line	25' below floodwall			BPU Drawings. Power Plant is moth-balled (inactive) Centerline Elev 751.7 - Drwg Sheet 2-3 (Contract No. K-76: Circulating Water Pipe Layout and Profile (Inf.)			
75+22	6"	KAW PP Intake Water	18' below floodwall	18' below floodwall					
75+32	72"	KAW PP Intake Influent #1	20' below floodwall			It appears that seepage rings were used on most of the pipes crossing the levee. EM 1110-2-1913 now states that seepage rings should not be provided. We will not be recommending removal of these seepage rings for Phase 2.			
75+45	3"	KAW PP Intake Chlorine	Chlorine Pipe Elev~ 761 - SP			Chlorine PVC pipe Elev 764.5' Info from BPU drawings: Sheet P48-2, Contract K-87: Chlorine Piping (1960)			
75+50	1" OR 2"	KAW PP Intake Gas		9' below levee / Eleve 763		Gas line also has a meter on it located under the bridge. The pipe penetrates the floodwall approximately 18" below the top of the floodwall.			
75+50	4"	KAW PP Conduit				Took picture of 4" conduit penetrating floodwall and leading to corrugated handhole at landside "toe" of floodwall fill material. Ernie states that conduit runs along toe of levee and ends at BPU "sealing weir" (discharge building)			
75+62	17"W X 43"H	KAW Power Plant Electrical		Elev. 762.5; 18" x 48" duct		Elev 756 according to BPU 1952 As-Built			
75+89	72"	KAW PP Intake Influent #2		Elev 742 / 30' below floodwall					

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Armourdale Utility Crossings: Inventory and Action for N500+3 Raise
Created by: Melissa Corkill
Peer Reviewed by: Hank Mildenberger

Date: 23-Aug-2006
Modified: 02-Mar-2007

UTILITY IDENTIFICATION			BACKGROUND DOCUMENTS AND RESEARCH NOTES FOR EACH UTILITY CROSSING								
75+90	2"	Water Emergency		Not shown		BPU Drawings: Normally not under pressure. Valve and 4' cover (in 1959 at lower levee elevation). Since levee was raised ~5', line may be 9' buried			
76+83	72"	Osage Pump Plant Outfall		30+ below levee				X	
79+60	84"	KAW PP Discharge		40+ below levee				X	
79+80	60"	KAW PP Discharge		40+ below levee				X	
81+07	2.5" in 4"	Calcium Carbonate	20' below levee	20' below levee. Drainage structure exists on drwy but no indication of closure device	Abandoned. Gate valve. Used to discharge calcium carbonate.			Not listed	
90+79	30"	Sanitary Sewer (Inverted)		Elev 731.3				X	
91+76	30"	Storm Sewer		Elev 732.2				X	
99+63	4"	Discharge Line	6' below levee	6' below levee		Could not find inlet. Did not go to river to find outlet. Likely that inlet is covered up and line is not in use.			
108+95	42"	Sanitary Sewer		42" RCP with Sluice Gate. Interceptor San. Sewer				X	
127+20	24"	Potable Water	20' below levee	20' below levee	Abandoned	Found control structure with valve. Appeared that everything was intact. Saw cuts in structure indicate some sort of work done in pit after construction. Should check with BPU about functionality	Appears on Wyandotte CAD files	X Abandoned	
129+20	5'X6'	12th St. Pump Plant Outfall		30' to 40' below levee				X	
129+20	20" x 2	12th St. Pump Plant Outfall							DRG Pump Station Record Drawings. Elev of DIP 767.24
129+60	72"	12th St. Storm Sewer	Elevation unknown, plugged to remain in place	not shown					Design Memo 1971: During 1952 construction, the original brick sewer from the PP to the river was plugged with concrete at each end, then filled with pervious and impervious material zoned with the restored levee section.
156+75	6'X6'	Mill St. Pump Plant Outfall				Discharge pipe is about 2' below levee.		X	No Flap Gate, Site Visit 1-9-07
156+75	20" x 2	Mill St. Pump Plant Outfall							
172+05	12"	KS Gas							KGE Maps, Date: 9-15-2005

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Armourdale Utility Crossings: Inventory and Action for N500+3 Raise
Created by: Melissa Corkill
Peer Reviewed by: Hank Mildenberger

Date: 23-Aug-2006
Modified: 02-Mar-2007

UTILITY IDENTIFICATION			BACKGROUND DOCUMENTS AND RESEARCH NOTES FOR EACH UTILITY CROSSING						
185+70	72"	5th St. Pump Plant Outfall						X	
186+74	30"	Sanitary Sewer	elevation unknown	Greystone Heights Sewer. Gate Elevation Unknown			48" DIP - 44' below ground	X	
194+60	18" and 12"	Midwest Cold Storage Pump Plant Outfall				Outlet was wet indicating it could still be used to drain interior drainage. Drainage structure did not appear to have any other contributing flows other than from Plant		X	Design Memo 1971: A 12" CIP was installed 6 feet above the 18" CIP between the landside and riverside gatewells for use in providing a connection to the drainage system for the ice plant condensers and pump.
212+76	24"	Storm Sewer						X	
220+64	24	Storm Sewer						X	
230+77	7.5' X 7.5'	Shawnee Ave Pump Plant Outfall		2 Sluice and 2 Flap				X	
231+38.91	57"	Storm Sewer	32' below levee - 57" concrete conduit, Old Shawnee Ave Pumping Plant Discharge (removed)	PLUGGED BOTH ENDS					
240+73	48"	Storm Sewer	Flapgate on outfall	sluice gate and flap gate				X	
244+70	24"	Storm Sewer		Elev 739.3				X	
246+53	30"	Storm Sewer Forcemain	15' below levee Elev 745	25' below levee. Elev 733.3				X Termination of RR Forcemain	
250+31	12"	Storm Sewer		Elev 735				X	
253+43	12"	Storm Sewer		Elev 736.4		KVDD stated it was grouted properly		X Abandoned	
256+71	12"	Storm Sewer	22' below levee	22' below levee Elev 736.5	Not listed			X Abandoned	
260+00	30"	Storm Sewer		Elev 725.4. Gate and Flap				X Slot Gate and Flap	
260+64	12"	Storm Sewer	20' below levee	Elev 737	Listed on HNTB's list. No indication of abandonment			X Slot Abandoned	
262+89	12"	Storm Sewer		Elev 736.8				X Gate Valve and Flap	
266+76	16"	Storm Sewer		Elev 736.4				X Slot, Sluice and Flap	
276+00	NA	Fiber Optic				Oct 24 site visit: fiber optic observed between bridges			
276+79	42"	Kansas City Southern Pump Plant Outfall		Elev 727.4				X	
281+50	42"	Storm Sewer		Invert Elev 728, Sluice and Flap	RCP. Pipe is under pressure due to flows coming from elevated interstate			X I-670 Storm Sewer	
286+59	24"	PBI Gordon Pump Plant Gravity Storm Flow		Elev 729.7				X	

Armourdale Utility Crossings: Inventory and Action for N500+3 Raise
Created by: Melissa Corkill
Peer Reviewed by: Hank Mildenberger

Date: 25-Aug-2006
Modified: 02-Mar-2007

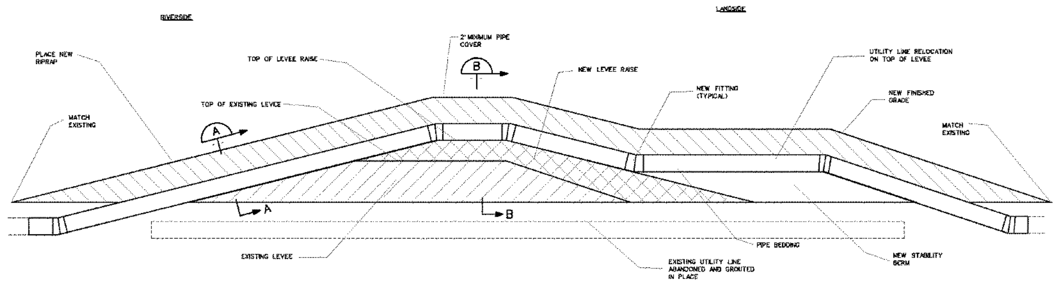
UTILITY IDENTIFICATION			BACKGROUND DOCUMENTS AND RESEARCH NOTES FOR EACH UTILITY CROSSING						
286+59	6"	PBI Gordon Pump Plant Discharge Pipe							
290+52	24"	Storm Sewer		Elev 724.6				X	
295+45	18"	Storm Sewer		Elev 735	18" RCP			X - Sluice Gate	
295+52	24"	National Beef Pumping Plant / KCK Pump Plant / (Central Ave. PP - KVDD)		Elev 731.7		Active: Water flowing during site visit. There are two 18" CIP pipes on the bridge		X City - Central Ave PS	
295+79	2"	National Beef Pumping Plant / KCK Pump Plant / (Central Ave. PP - KVDD)	Elev 753.4, Penetrates Floodwall						
295+80	10"	National Beef Pumping Plant / KCK Pump Plant / (Central Ave. PP - KVDD)	Elev 753.6, Penetrates Floodwall						
299+20	3"	Central Ave. Pumping Plant - Sump Pump		Elev 757.7					
299+20	2-14"	Central Ave. Pump Plant - Discharge						X - City - 2 flaps, 1 gate	
311+11	7.5' X 7.5'	Storm Sewer		only sluice gate, Elev 727.4	Called the split log outfall. Sluice Gate			X 7.5 x 7.5 City	
315+10	5' X 4'	Storm Sewer		only sluice gate, Elev 735.3	This is possibly the outfall for the Fowler Street Sewer System			X - 5x5	
324+58 (41+45 LE)	48"	Sanitary Force Main	Line not present	Gate Valve, Elev 740	Pressure Sewer			Gate Valve	Invert Elev ~ 740 (Armourdale Interceptor Drawings)
328+18 (45+05 LE)	42"	Storm Sewer/Sanitary Sewer		No sluice gate or flap. Invert Elev 732.5	Sluice Gate		Wyandotte County maps show this as a CSO.	Not listed	No gate in structure per gatewell inventory table
329+35 (46+22 L.E.)	24"	Storm Sewer	No sluice gate indicated	Invert Elev. 753.8 No sluice gate or flap indicated	Flap Gate			Not listed	
50+00	8"	KS Gas						Not listed	KGS Maps
334+08 (50+95 LE)	24"	Storm Sewer	24" CMP	24" CMP Invert Elev. 753.6 No sluice gate or flap indicated	Flap Gate			Not listed	
334+83 (51+70 L.E.)	12"	Spring Sump Drain	12" spring sump drain	12" spring sump drain	Not included in list			Not listed	

Armourdale Utility Crossings: Inventory and Action for N500+3 Raise
Created by: Melissa Corkill
Peer Reviewed by: Hank Mildenberger

Date: 23-Aug-2006
Modified: 02-Mar-2007

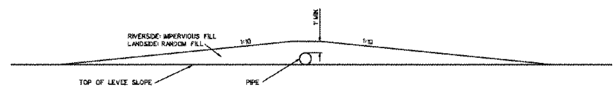
UTILITY IDENTIFICATION			BACKGROUND DOCUMENTS AND RESEARCH NOTES FOR EACH UTILITY CROSSING						
339+54 (56+41 L.E)	24"	Storm Sewer	No sluice gate or flap	No sluice gate or flap	Flap Gate, No Sluice Gate Required due to high ground		Storm Pipe Shown	Not listed	
339+63 (56+60 LE)	1 - 36"	Sanitary Force Main	No line exists	no sewer exists	Forcemain, River Crossing		1 - 36"	X	1963 Drawings show Armourdale Interceptor Forcemain
340+49 (57+36 LE)	12"	Storm Sewer	Working not plugged	Active, Elev 755.5	Listed on HNTB's list only because of its inclusion in the inventory from previous feasibility phase. List indicates flap gate present. HNTB indicates only 2 pipes operational in this area.		Storm Pipe Shown	Not listed	
341+43 (58+30 LE)	25" X 16"	Storm Sewer	Working not plugged	Plugged, elev 755	Not included in list		Storm Pipe Shown	Not listed	
343+59 (60+45 LE)	25" X 16"	Storm Sewer	No sluice gate or flap gate	Invert Elev 755	Flap Gate, No Sluice Gate Required due to high ground		Storm Pipe Shown	Not listed	
LE		Oxygen	Not on drawings	Not on drawings	Not listed				

Exhibit A-6.5: Typical Utility Crossings



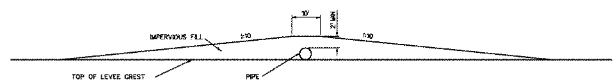
A TYPICAL UTILITY CROSSING

SCALE: NTS



A RIVERSIDE AND LANDSIDE SLOPE

SCALE: NTS



B LEVEE CREST

SCALE: NTS

NOTES:
1. IF PERMISSION IS GRANTED FOR A PIPELINE TO CROSS THE LEVEE BELOW THE TOP OF LEVEE, A POSITIVE CUTOFF VALVE OR GATE CLOSURE WILL BE INSTALLED AT THE RIVERSIDE SHOULDER OF THE LEVEE ROAD.

US Army Corps of Engineers
Kansas City District

Symbol	Description	Date	Appr.

Designed by: MRC	Date: MARCH 2007	U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI
Drawn by: MRC	Drawing code:	
Checked by: HMW	Plot scale: NA	
Submitted by: HMW	CADD File Name: TYPICALUTILITYCROSS...	

KANSAS CITY LEVEES
FEASIBILITY LEVEL DESIGN

TYPICAL UTILITY CROSSING FOR LEVEE RAISE

Sheet Reference Number:
XX

EXHIBIT A-6.6

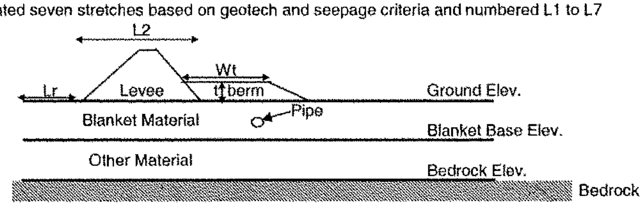
Armourdale Utility Uplift Spreadsheet: Data Entry Worksheet

Armourdale Utility Uplift Spreadsheet: Data Entry Worksheet
Kansas City Levees Phase 2

Created By:	Melissa Corkill	Date:	11-Aug-06
		Date Modified:	6-Mar-07
Peer Reviewed By:		Date:	

How to Use this Spreadsheet

Insert parameters into cells highlighted in orange and the corresponding spreadsheets will update automatically.
Spreadsheets are linked, therefore spreadsheets must stay in their original directories to maintain links.
Armourdale Unit was separated seven stretches based on geotech and seepage criteria and numbered L1 to L7



Levee Type Number		L1	L2	L3	L4	L5	L6	L7
Station Start		5+00UE	66+00	79+00	190+00	254+00	296+00	313+00
Station End		66+00	79+00	190+00	254+00	296+00	313+00	40+00LE
Levee Width, L2 (from geotech)		100		100		50		100
Riverside Blanket Width, Lr (from		200		65		100		110
Top of Levee Elev. (n500 + 3ft)		777.5	777	775	769.5	765.8	763	762
Berm Width for the w/ Berm option, Wt (ft) (from geotech)		NA	NA	NA	NA	NA	NA	NA
Berm Height for the w/Berm option, t (feet) (from geotech)		NA	NA	NA	NA	NA	NA	NA
Ground Elev. Landside		762	760	760	751	750	747	750
Blanket Base Elev.		740	748	730	728	725	729	725
Bedrock Elev. (from O&M manual Borings)		670	671	667	668	678	678	700
Pipe Depth ft. (enter on spreadsheet		Varies	Varies	Varies	Varies	Varies	Varies	Varies
Pipe Diameter in. (enter on spreadsheet)		Varies	Varies	Varies	Varies	Varies	Varies	Varies
Blanket Soil Type (from geotech)		ML				ML-CL		ML-CL
Blanket Soil Unit Weight, soil density (pcf)		115	115	115	115	115	115	115
Blanket Thickness Dbo ft (from geotech EC-GD) landside		22		30		25		25
Depth of Sands Df ft (from Geotech EC-GD)		70		63		47		25
Max Head or Levee Height ft (from geotech EC-GD)		15.5	17.0	15.0	18.5	15.8	16.0	12.0
Notes			Cutoff Wall		Relief Wells		Relief Wells	

Input Data	
Calculated	
NA	Not Applicable

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NOMENCLATURE for all Uplift Spreadsheets

Input

(Kf/Kb)R = riverside permeability
(Kf/Kb)L = landside permeability
DbL = landside blanket thickness
Dbr = riverside blanket thickness
Dbo = levee toe blanket thickness
Df = thickness of pervious foundation
Lr = length of riverside blanket
LL = length of landside blanket
H = max head or levee height
L2 = levee base width
t = berm height, ft
ground elevation = average elevation of landside ground

Output

Cr = riverside effective length coefficient
CL = landside effective length coefficient
where C = [(Kf/Kb) * Df * Db]1/2
L1 = riverside effective length
where L1 = C * (e (2LR/C-1)) / (e(2LR/C+1))
Le = landside effective length
Lt = total effective length
ho = head above tailwater at levee toe
io = seepage gradient
ic = critical gradient = (γsat-γwater)/γwater

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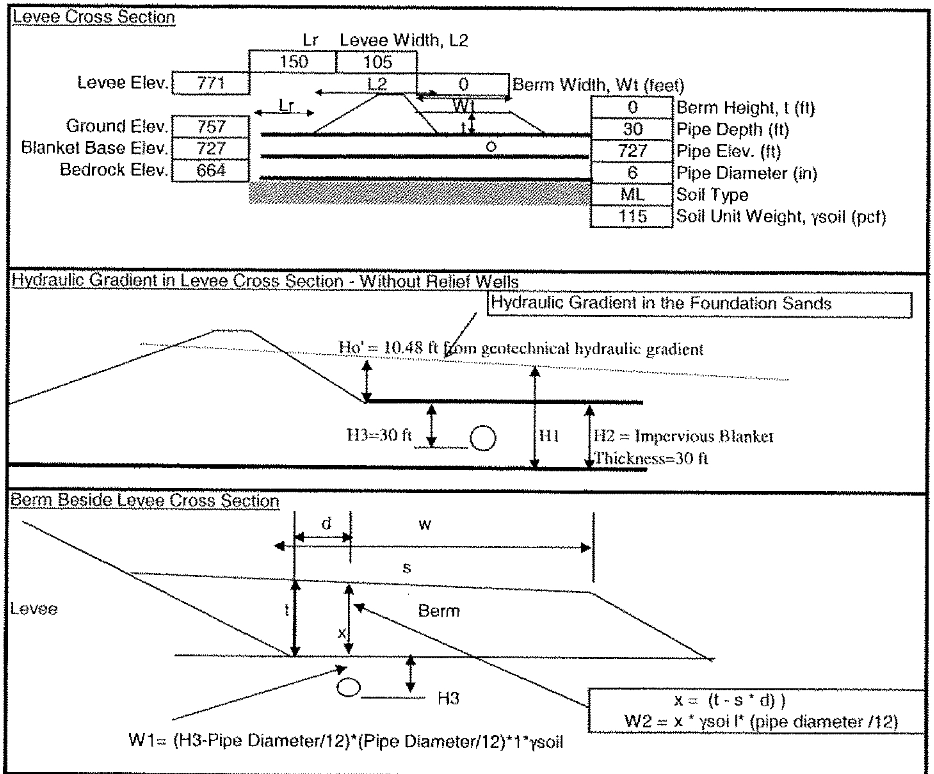
EXHIBIT A-6.7

Sample Calculation for Utility Uplift

Kansas City's Levees - Phase 2
Armourdale - Utility Uplift Sample Calculation at Station 200+00 (Distance from toe - 0ft)

By: Melissa Corkill	Date Created:	14-Aug-06
	Date Modified:	15-Aug-06
Peer Reviewed By: Scott Loehr	Date Reviewed:	15-Aug-06



Definitions

H1 = Height of Hydraulic Gradient above base of Impervious Blanket - ft
H2 = Impervious Blanket Thickness - ft
H3 = Depth of Pipe Invert -ft
Ho' = Excess head above ground surface (initial Ho' is calculated at toe of levee by geotechnical engineer) - ft
Lr = Length of riverside blanket (determined by geotechnical engineer) - ft
L1 = Riverside effective length (calculated by geotechnical engineer) - ft
L2 = Levee Width (toe to toe) - ft
Wt = Berm Width - ft
t = Berm Height at toe of levee- ft

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Page 1 of 2

x = Berm Height at structure (pipe)- ft
s = slope of berm
Ws = weight of structure (pipe) per foot of length
18.97 lb per ft (6" Diameter Steel Pipe, .280" wall thickness, Manual of Steel Construction)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((3/12)^2) *1*62.4 = 12 lb/lf
S = surcharge loads = weight of saturated soils above structure = W1 + W2
W1 = Surcharge load above structure (not including berm)
W2 = Surcharge load of berm above structure
P1 = Pressure at the base of the impervious blanket at the location of the structure being investigated
P3 = Pressure at the base of the structure (pipe) being investigated
U = Uplift force on the project area of structure = Area of pipe * P3
Wg = weight of surcharge water above top surface of structure control by gravity flow = 0 for pipes
SFf = Flotation Safety Factor = (Ws+Wc+S)/(U-Wg)

Sample Calculations

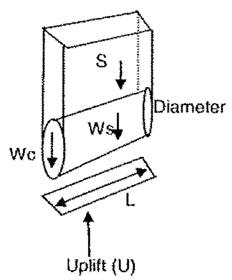
Sample Calculations are done at the toe of the levee (distance from toe = 0 ft)

H1

H1 = Ho' + Ground Elev - Impervious Blanket Base Elev
H1 = 10.48 ft + 757 ft - 727 ft = 40.48 ft

P3

P3 = H3 x (H1/H2) x γ_{water}
P3 = 30 ft x (40.48ft /30ft) x 62.4 pcf = 2525.9 psf



Wc

Wc = pi x r^2 x γ_{water}
Wc = 3.1416 x ((3"/12)^2) x 62.4 pcf = 12 lb/lf

S

S = W1 + W2
W1 = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
W1 = (30 ft - 6"/12) x (6"/12) x 115 pcf x 1 lf
W1 = 1696 pounds
W2 = (t - s x d) (Pipe Diameter/12) x 1 lf x γsoil
W2 = (5 - (0.05 x 0))x (6/12 x 1 lf x 115 psf) = 287.5 pounds
S = W1 + W2 = 1696 + 287.5 = 1983.5 pounds

U

U = Area of pipe * P3 = (Pipe Diameter/12) x 1 ft x P3
U = 6"/12 x 2525.9 psf x 1 ft
U = 1263 lb

SFf (Pipe Full)
SFf = Flotation Safety Factor = (Ws+Wc+S)/(U-Wg)
SFf = (19 + 12 + 1696)/(1263-0)
SFf = 1.37

SFf (Pipe Empty)
SFf = (19 + 1696)/(1263-0)
SFf = 1.36

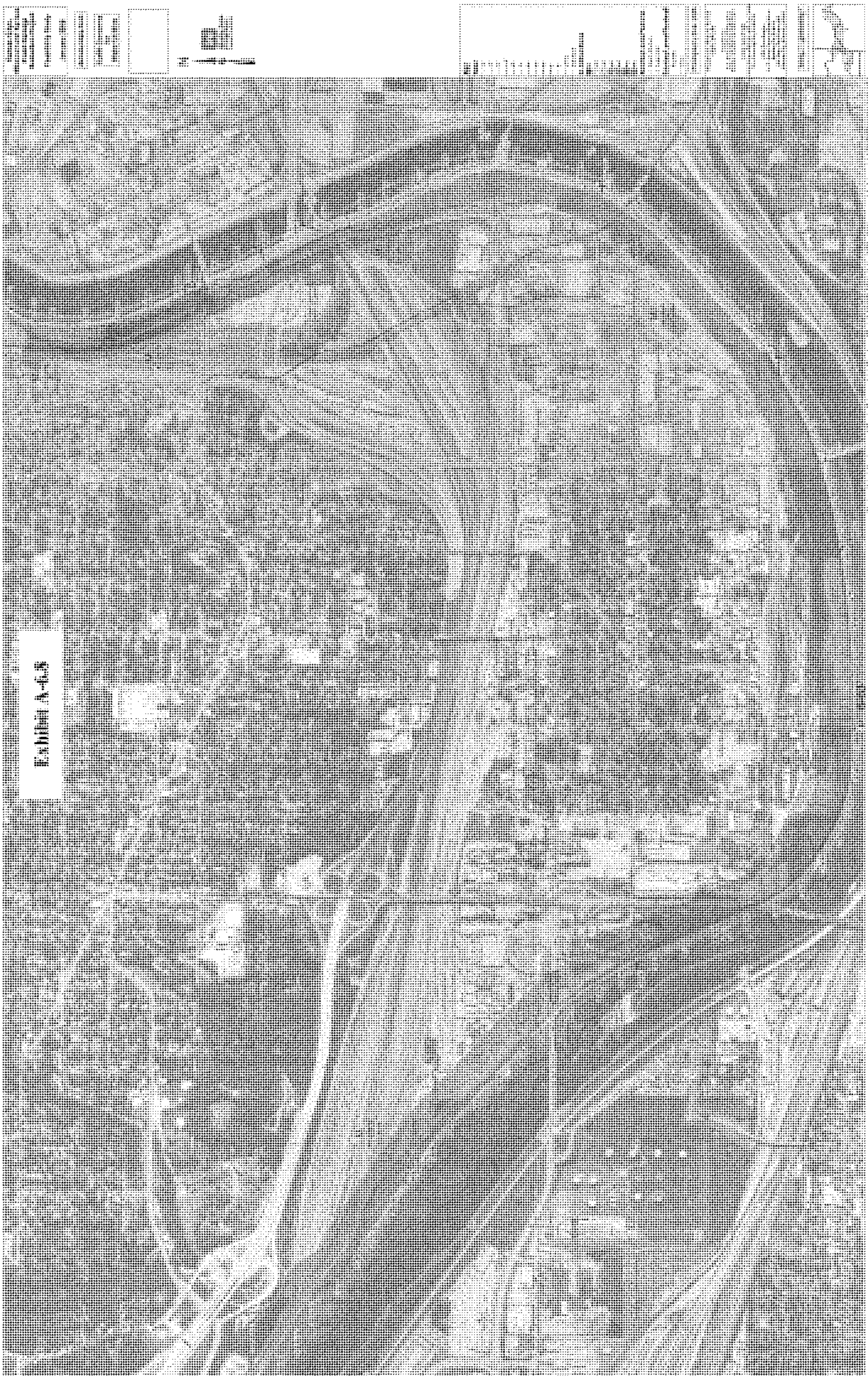


Exhibit A-6.5

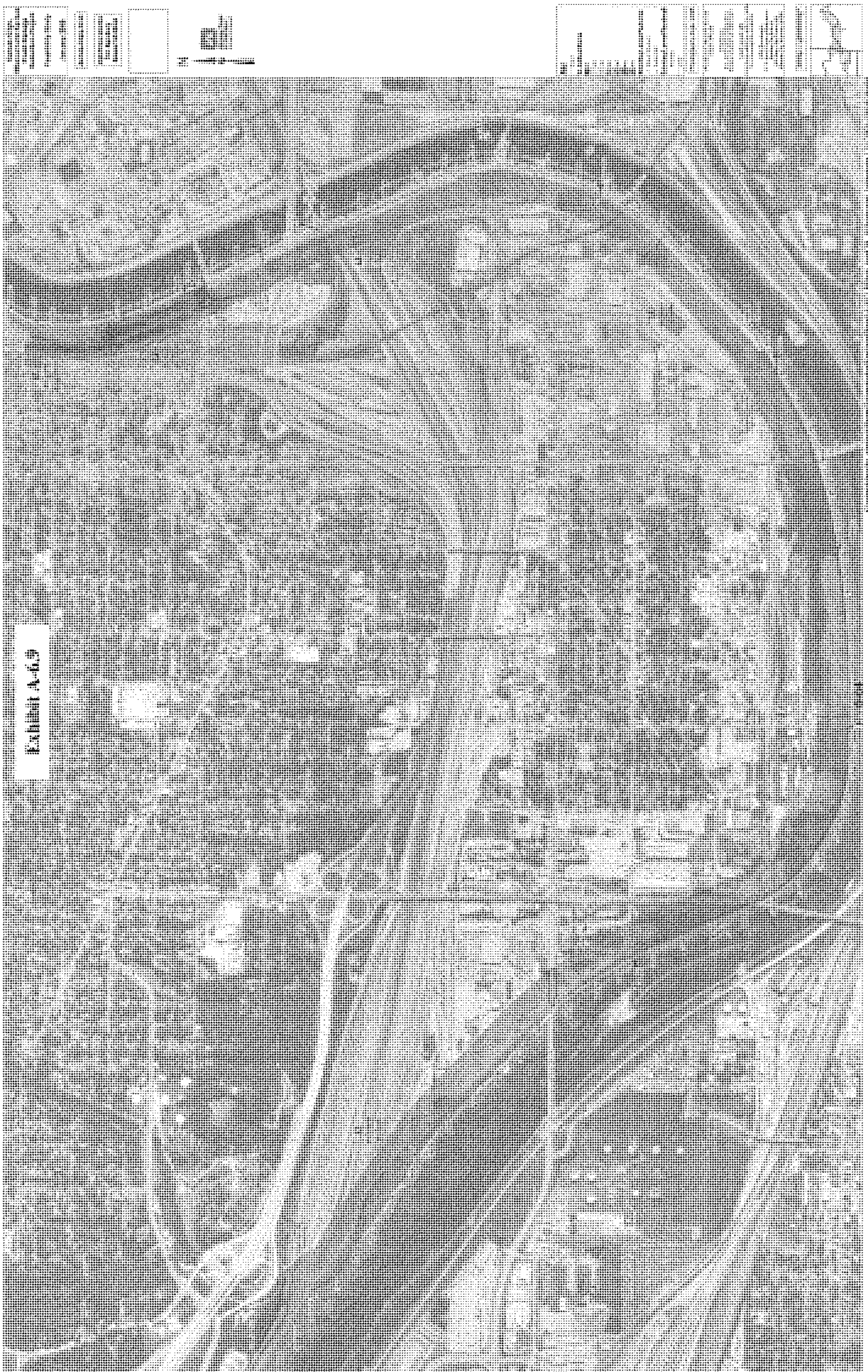


Exhibit A-6.9

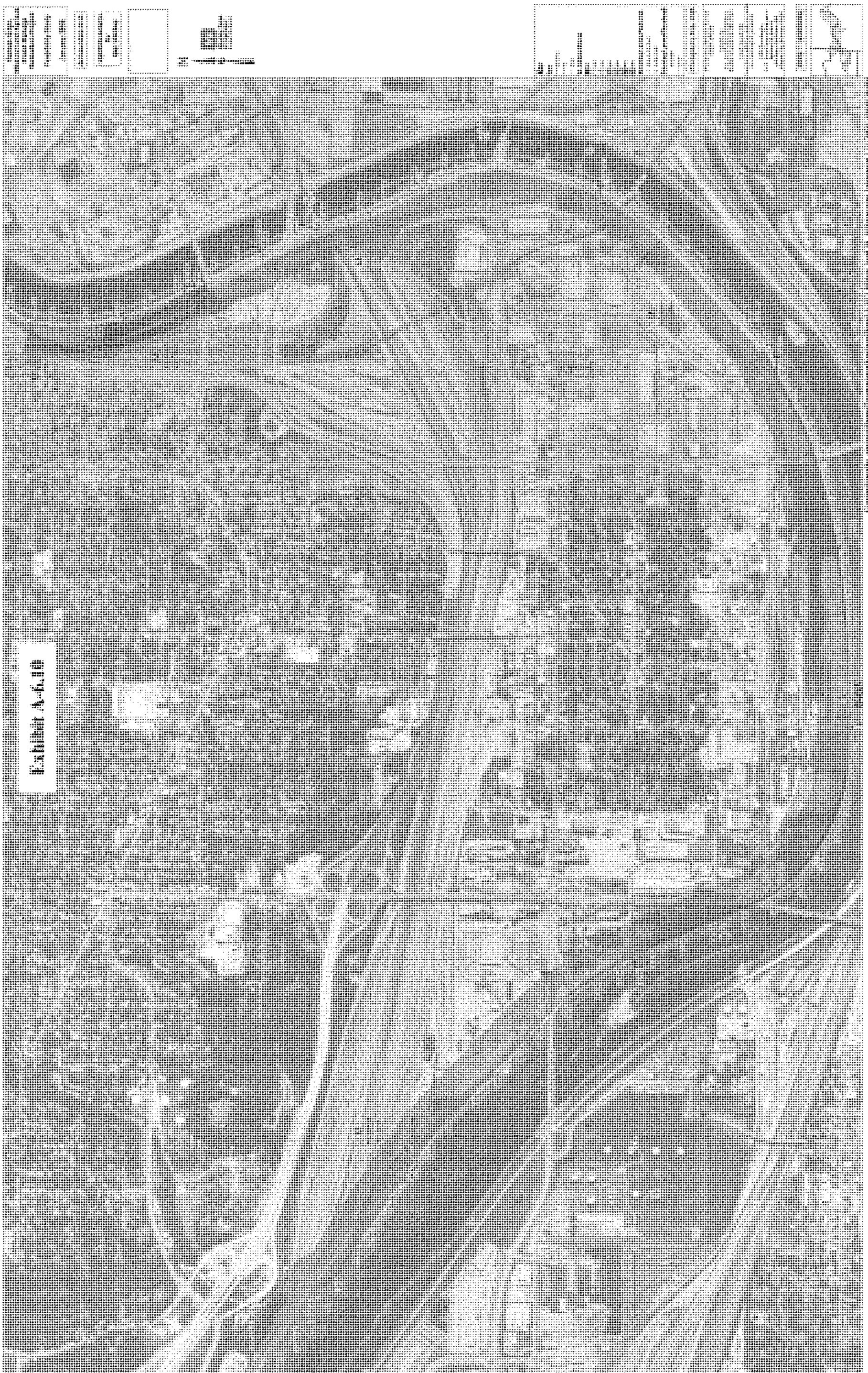


Exhibit A-6.10

EXHIBIT A-6.11

N500+3 Utility Uplift Calculations

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
5+00UE	300	300	22	22	22	70	15.5	0.84	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.000
66+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

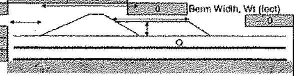
Lr 200 | Levee Width, L2 100

Levee Elev. 777.5

Ground Elev. 762

Blanket Base Elev. 740

Bedrock Elev. 670



0 Berm Width, Wt (feet)

0 Berm Height, t (feet)

3.33 Pipe Depth (ft)

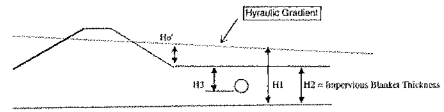
758.67 Pipe Elev.

6 Pipe Diameter (in)

ML Soil Type

115 Soil Unit Weight, γsoil (pcf)

3.33 Pipe Depth (Check)



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 6 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	758.67	740	762	3.33	22	32.8	309.9	19	12	163	155	0	1.25	1.17
25	10.42	758.67	740	762	3.33	22	32.4	306.3	19	12	163	153	0	1.27	1.19
50	10.05	758.67	740	762	3.33	22	32.0	302.7	19	12	163	151	0	1.28	1.20
100	9.34	758.67	740	762	3.33	22	31.3	296.0	19	12	163	148	0	1.31	1.23
125	9.00	758.67	740	762	3.33	22	31.0	292.8	19	12	163	146	0	1.32	1.24
150	8.67	758.67	740	762	3.33	22	30.7	289.7	19	12	163	145	0	1.34	1.25
200	8.06	758.67	740	762	3.33	22	30.1	283.9	18	12	163	142	0	1.37	1.28
300	6.96	758.67	740	762	3.33	22	29.0	273.5	19	12	163	137	0	1.42	1.33
400	6.00	758.67	740	762	3.33	22	28.0	264.5	19	12	163	132	0	1.47	1.37
500	5.18	758.67	740	762	3.33	22	27.2	256.7	19	12	163	128	0	1.51	1.42

Ws Calc 6 -in steel 1ft
19
Wc Calc 12

See Sample Calculations for list of abbreviations and sample calculations

Ws = weight of structure per foot of length = 18.97 lb per ft (6-inch Diameter Steel Pipe, .280" thick)

Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((3/12)^2) *1*62.4 = 12 plf

S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil

P3 = H3*(H1/H2)*γwater

U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3

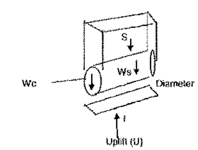
Wg = weight of surcharge water above top surface of structure control by gravity flow

SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is met

Enter the Distance from toe and change the value until the SFf is equal to that required

Ground Elev 762	Distance from toe (ft) 0				Uplift Safety Extreme Case (1.1 REQ'D)	
	Ws = 19		Wc = 12 Full			
Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
762	0		19	12		
758.67	163	155	19	12	1.25	1.17
757.67	220	202	19	12	1.25	1.19
756.67	278	248	19	12	1.25	1.20
755.67	335	295	19	12	1.24	1.20
754.67	393	341	19	12	1.24	1.21
753.67	450	388	19	12	1.24	1.21
752.67	508	434	19	12	1.24	1.21
751.67	565	481	19	12	1.24	1.22
750.67	623	527	19	12	1.24	1.22



U = P3 * diameter * l

Using l = 1

Conclusions

For levee stretch: 5+00UE to 66+00

For pipe diam (in): 6

There is no uplift concern at a depth of 40 inches (3.3 feet) or greater within 500 feet landward of the levee

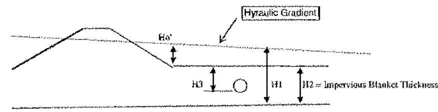
Armourdale Area L1 Stationing Range	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t (feet)	s
5+00UE	300	300	22	22	22	70	15.5	0.84	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.000

66+00
500-yr + 3ft

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.

Lr 200 Levee Width, L2 100

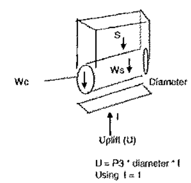
Levee Elev. 777.5
Berm Width, Wt (feet) 0
Berm Height, t (feet) 0
Ground Elev. 762
Blanket Base Elev. 740
Bedrock Elev. 670
3.33 Pipe Depth (ft)
758.67 Pipe Elev.
12 Pipe Diameter (in)
ML Soil Type
115 Soil Unit Weight, γsoil (pcf)
3.33 Pipe Depth (Check)



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	758.67	740	762	3.33	22	32.8	309.9	50	49	268	310	0	1.18	1.02
25	10.42	758.67	740	762	3.33	22	32.4	306.3	50	49	268	306	0	1.20	1.04
50	10.05	758.67	740	762	3.33	22	32.0	302.7	50	49	268	303	0	1.21	1.05
100	9.34	758.67	740	762	3.33	22	31.3	296.0	50	49	268	296	0	1.24	1.07
125	9.00	758.67	740	762	3.33	22	31.0	292.8	50	49	268	293	0	1.25	1.08
150	8.67	758.67	740	762	3.33	22	30.7	289.7	50	49	268	290	0	1.27	1.10
200	8.06	758.67	740	762	3.33	22	30.1	283.9	50	49	268	284	0	1.29	1.12
300	6.96	758.67	740	762	3.33	22	29.0	273.5	50	49	268	273	0	1.34	1.16
400	6.00	758.67	740	762	3.33	22	28.0	264.5	50	49	268	265	0	1.36	1.20
500	5.18	758.67	740	762	3.33	22	27.2	256.7	50	49	268	257	0	1.43	1.24

Ws Calc 12-in steel 1ft
50
Wc Calc 49

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 50 lb per ft (12-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((6/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

Ground Elev 762		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
	Ws = 50		Wc = 49 Full			
Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
762	0		50	49		
758.67	268	310	50	49	1.18	1.02
757.67	383	403	50	49	1.19	1.07
756.67	498	496	50	49	1.20	1.10
755.67	613	589	50	49	1.21	1.12
754.67	728	682	50	49	1.21	1.14
753.67	843	775	50	49	1.21	1.15
752.67	958	868	50	49	1.22	1.16
751.67	1073	961	50	49	1.22	1.17
750.67	1188	1055	50	49	1.22	1.17

Conclusions
For levee stretch: 5+00UE to 66+00
For pipe diam (in): 12
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 200 feet landward of the levee
There is uplift concern at a depth of 4.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Ci	t	s
L1																							feet	
Stationing Range																								
5+00UE	300	300	22	22	22	70	15.5	0.84	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.009
66+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr Levee Width, L2

200 100

Levee Elev. 777.5

0 Berm Width, Wt (feet)

0 Berm Height, t (feet)

Ground Elev. 762

Blanket Base Elev. 740

Bedrock Elev. 670

10 Pipe Depth (ft)

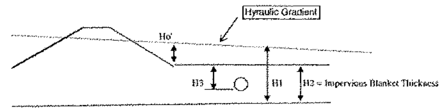
752 Pipe Elev.

12 Pipe Diameter (in)

ML Soil Type

115 Soil Unit Weight, γsoil (pcf)

10 Pipe Depth (Check)



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	752	740	762	10	22	32.8	930.8	50	49	1035	931	0	1.22	1.17
25	10.42	752	740	762	10	22	32.4	919.7	50	49	1035	920	0	1.23	1.18
50	10.05	752	740	762	10	22	32.0	909.0	50	49	1035	909	0	1.25	1.19
100	9.34	752	740	762	10	22	31.3	888.8	50	49	1035	889	0	1.28	1.22
125	9.00	752	740	762	10	22	31.0	879.2	50	49	1035	879	0	1.29	1.23
150	8.67	752	740	762	10	22	30.7	870.0	50	49	1035	870	0	1.30	1.25
200	8.06	752	740	762	10	22	30.1	852.6	50	49	1035	853	0	1.33	1.27
300	6.96	752	740	762	10	22	29.0	821.3	50	49	1035	821	0	1.38	1.32
400	6.00	752	740	762	10	22	28.0	794.3	50	49	1035	794	0	1.43	1.37
500	5.18	752	740	762	10	22	27.2	771.0	50	49	1035	771	0	1.47	1.41

Ws Calc
12-in steel 1lf
50
Wc Calc
49

See Sample Calculations for list of abbreviations and sample calculations

Ws = weight of structure per foot of length = 50 lb per ft (12-inch Diameter Steel Pipe, .375" thick)

Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((6/12)^2) *1*62.4 = 49 plf

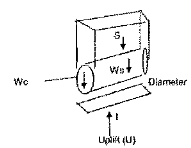
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil

P3 = H3*(H1/H2)*γwater

U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3

Wg = weight of surcharge water above top surface of structure control by gravity flow

SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l

Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is met

Enter the Distance from toe and change the value until the SFf is equal to that required

Ground Elev 762		Distance from toe (ft) 0				
	Ws = 50		Wc = 49 Full		Uplift Safety Extreme Case (1.1 REQ'D)	
Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
762	0		50	49		
752	1035	931	50	49	1.22	1.17
751	1150	1024	50	49	1.22	1.17
750	1265	1117	50	49	1.22	1.18
749	1380	1210	50	49	1.22	1.18
748	1495	1303	50	49	1.22	1.19
747	1610	1396	50	49	1.22	1.19
746	1725	1489	50	49	1.22	1.19
745	1840	1582	50	49	1.23	1.19
744	1955	1675	50	49	1.23	1.20

Conclusions

For levee stretch: 5+00UE to 66+00

For pipe diam (in): 12

There is no uplift concern at a depth of 10 feet or deeper within 500 feet landward of the levee

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Lc	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
5+00UE	300	300	22	22	22	70	15.5	0.64	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.000
66+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr Levee Width, L2

200 100

Levee Elev. 777.5

0 Berm Width, Wt (feet)

0 Berm Height, t (feet)

Ground Elev. 762

Blanket Base Elev. 740

Bedrock Elev. 670

3.33 Pipe Depth (ft)

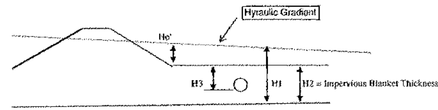
758.67 Pipe Elev.

16 Pipe Diameter (in)

ML Soil Type

115 Soil Unit Weight, γsoil (pcf)

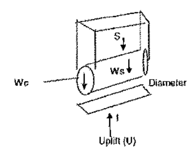
3.33 Pipe Depth (Check)



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 16 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	758.67	740	762	3.33	22	32.8	309.9	63	87	306	413	0	1.10	0.89
25	10.42	758.67	740	762	3.33	22	32.4	306.3	63	87	306	408	0	1.12	0.90
50	10.05	758.67	740	762	3.33	22	32.0	302.7	63	87	306	404	0	1.13	0.91
100	9.34	758.67	740	762	3.33	22	31.3	296.0	63	87	306	395	0	1.16	0.94
125	9.00	758.67	740	762	3.33	22	31.0	292.8	63	87	306	390	0	1.17	0.95
150	8.67	758.67	740	762	3.33	22	30.7	289.7	63	87	306	386	0	1.18	0.96
200	8.06	758.67	740	762	3.33	22	30.1	283.9	63	87	306	379	0	1.21	0.98
300	6.96	758.67	740	762	3.33	22	29.0	273.5	63	87	306	365	0	1.25	1.01
400	6.00	758.67	740	762	3.33	22	28.0	264.5	63	87	306	353	0	1.28	1.05
500	5.18	758.67	740	762	3.33	22	27.2	256.7	63	87	306	342	0	1.33	1.08

Ws Calc
16 -in steel 1lf
63
Wc Calc
87

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 62.58 lb per ft (16-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((8/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 762		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 63		Wc = 87 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	762	0		63	87		
3.33	758.67	306	413	63	87	1.10	0.89
4.33	757.67	459	537	63	87	1.13	0.97
5.33	756.67	613	661	63	87	1.15	1.02
6.33	755.67	766	786	63	87	1.17	1.06
7.33	754.67	919	910	63	87	1.18	1.08
8.33	753.67	1073	1034	63	87	1.18	1.10
9.33	752.67	1226	1158	63	87	1.19	1.11
10.33	751.67	1379	1282	63	87	1.19	1.13
11.33	750.67	1533	1406	63	87	1.20	1.13

Conclusions
For levee stretch: 5+00UE to 66+00
For pipe diam (in): 16
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

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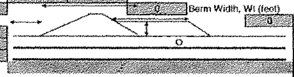
Page 1 of 3

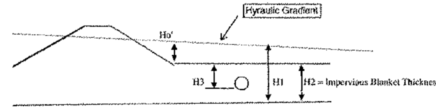
MARCH 8, 2007

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Lc	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
5+00UE	300	300	22	22	22	70	15.5	0.84	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.000
66+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr Levee Width, L2

Levee Elev.	777.5	
Lr	200	
Levee Width, L2	100	
Berm Width, Wt (feet)	0	
Berm Height, t (feet)	0	
Ground Elev.	762	
Blanket Base Elev.	740	
Bedrock Elev.	670	
Pipe Depth (ft)	10	
Pipe Elev.	752	
Pipe Diameter (in)	16	
Soil Type	ML	
Soil Unit Weight, γsoil (pcf)	115	
Pipe Depth (Check)	10	

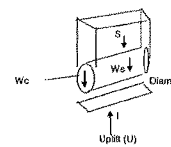




d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 16 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	752	740	762	10	22	32.8	930.8	63	87	1329	1241	0	1.19	1.12
25	10.42	752	740	762	10	22	32.4	919.7	63	87	1329	1226	0	1.21	1.14
50	10.05	752	740	762	10	22	32.0	909.0	63	87	1329	1212	0	1.22	1.15
100	9.34	752	740	762	10	22	31.3	888.8	63	87	1329	1185	0	1.25	1.17
125	9.00	752	740	762	10	22	31.0	879.2	63	87	1329	1172	0	1.26	1.19
150	8.67	752	740	762	10	22	30.7	870.0	63	87	1329	1160	0	1.27	1.20
200	8.06	752	740	762	10	22	30.1	852.6	63	87	1329	1137	0	1.30	1.22
300	6.96	752	740	762	10	22	29.0	821.3	63	87	1329	1095	0	1.35	1.27
400	6.00	752	740	762	10	22	28.0	794.3	63	87	1329	1059	0	1.40	1.31
500	5.18	752	740	762	10	22	27.2	771.0	63	87	1329	1028	0	1.44	1.35

Ws Calc
16 -in steel 1ft
63
Wc Calc
87

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 62.58 lb per ft (16-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((6/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of saluarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 762		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 63		Wc = 87 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	762	0		63	87		
10	752	1329	1241	63	87	1.19	1.12
11	751	1482	1365	63	87	1.20	1.13
12	750	1636	1489	63	87	1.20	1.14
13	749	1789	1613	63	87	1.20	1.15
14	748	1942	1737	63	87	1.20	1.15
15	747	2096	1862	63	87	1.21	1.16
16	746	2249	1986	63	87	1.21	1.16
17	745	2402	2110	63	87	1.21	1.17
18	744	2556	2234	63	87	1.21	1.17

Conclusions
For levee stretch: 5+00UE to 66+00
For pipe diam (in): 16
There is no uplift concern at a depth of 10 feet or deeper within 500 feet landward of the levee

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
5+00UE	300	300	22	22	22	70	15.5	0.84	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.000
66+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 200, Levee Width, L2 100

Levee Elev. 777.5

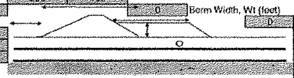
Berm Width, Wt (feet) 0

Berm Height, t (feet) 0

Ground Elev. 762

Blanket Base Elev. 740

Bedrock Elev. 670



15 Pipe Depth (ft)

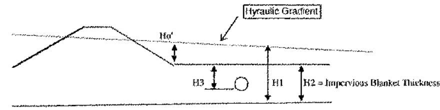
747 Pipe Elev.

16 Pipe Diameter (in)

ML Soil Type

115 Soil Unit Weight, γsoil (pcf)

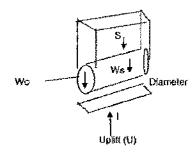
15 Pipe Depth (Check)



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 16 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	747	740	762	15	22	32.8	1396.1	63	87	2096	1862	0	1.21	1.16
25	10.42	747	740	762	15	22	32.4	1379.5	63	87	2096	1839	0	1.22	1.17
50	10.05	747	740	762	15	22	32.0	1363.5	63	87	2096	1816	0	1.24	1.19
100	9.34	747	740	762	15	22	31.3	1333.2	63	87	2096	1776	0	1.26	1.21
125	9.00	747	740	762	15	22	31.0	1318.8	63	87	2096	1758	0	1.28	1.23
150	8.67	747	740	762	15	22	30.7	1305.0	63	87	2096	1740	0	1.29	1.24
200	8.06	747	740	762	15	22	30.1	1278.8	63	87	2096	1705	0	1.32	1.27
300	6.96	747	740	762	15	22	29.0	1231.9	63	87	2096	1643	0	1.37	1.31
400	6.00	747	740	762	15	22	28.0	1191.5	63	87	2096	1589	0	1.41	1.36
500	5.18	747	740	762	15	22	27.2	1156.5	63	87	2096	1542	0	1.46	1.40

Ws Calc 16 -in steel 1ll
63
Wc Calc
87

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 62.58 lb per ft (16-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((8/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 762	Ws = 63	Distance from toe (ft) 0	Wc = 87 Full		Uplift Safety Extreme Case (1.1 REQ'D)	
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	762	0		63	87		
15	747	2096	1862	63	87	1.21	1.16
16	746	2249	1986	63	87	1.21	1.16
17	745	2402	2110	63	87	1.21	1.17
18	744	2556	2234	63	87	1.21	1.17
19	743	2709	2358	63	87	1.21	1.18
20	742	2862	2482	63	87	1.21	1.18
21	741	3016	2606	63	87	1.21	1.18
22	740	3169	2730	63	87	1.22	1.18
23	739	3322	2854	63	87	1.22	1.19

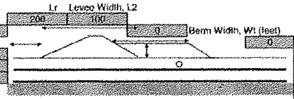
Conclusions
For levee stretch: 5+00UE to 66+00
For pipe diam (in): 16
There is no uplift concern at a depth of 10 feet or deeper within 500 feet landward of the levee

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Lc	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
5+00UE	300	300	22	22	22	70	15.5	0.84	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.000
66+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500 yr + 3ft																								

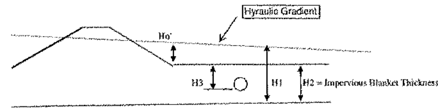
Lr Levee Width, L2

Levee Elev.	777.5
Ground Elev.	762
Blanket Base Elev.	740
Bedrock Elev.	670

200 100 0 Berm Width, Wt (feet)



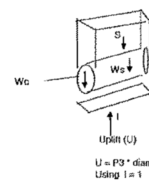
0	Berm Height, t (feet)
3.33	Pipe Depth (ft)
758.67	Pipe Elev.
24	Pipe Diameter (in)
ML	Soil Type
115	Soil Unit Weight, γsoil (pcf)
3.33	Pipe Depth (Check)



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 24 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	758.67	740	762	3.33	22	32.8	309.9	95	196	306	620	0	0.96	0.65
25	10.42	758.67	740	762	3.33	22	32.4	306.3	95	196	306	613	0	0.97	0.65
50	10.05	758.67	740	762	3.33	22	32.0	302.7	95	196	306	605	0	0.99	0.65
100	9.34	758.67	740	762	3.33	22	31.3	296.0	95	196	306	592	0	1.01	0.68
125	9.00	758.67	740	762	3.33	22	31.0	292.8	95	196	306	586	0	1.02	0.68
150	8.67	758.67	740	762	3.33	22	30.7	289.7	95	196	306	579	0	1.03	0.69
200	8.06	758.67	740	762	3.33	22	30.1	283.9	95	196	306	568	0	1.05	0.71
300	6.96	758.67	740	762	3.33	22	29.0	273.5	95	196	306	547	0	1.09	0.73
400	6.00	758.67	740	762	3.33	22	28.0	264.5	95	196	306	529	0	1.13	0.76
500	5.18	758.67	740	762	3.33	22	27.2	256.7	95	196	306	513	0	1.16	0.78

Ws Calc
24 -in steel 1ft
95
Wc Calc
196

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 94.62 lb per ft (24-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((12/12)^2) *1*62.4 = 196 plf
S = surcharge loads = weight of saluarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 762		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 95		Wc = 196 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	762	0		95	196		
3.33	758.67	306	620	95	196	0.96	0.65
4.33	757.67	536	806	95	196	1.03	0.78
5.33	756.67	766	992	95	196	1.07	0.87
6.33	755.67	996	1178	95	196	1.09	0.93
7.33	754.67	1226	1364	95	196	1.11	0.97
8.33	753.67	1456	1551	95	196	1.13	1.00
9.33	752.67	1686	1737	95	196	1.14	1.03
10.33	751.67	1916	1923	95	196	1.15	1.05
11.33	750.67	2146	2109	95	196	1.16	1.06
12.33	749.67	2376	2295	95	196	1.16	1.08
13.33	748.67	2606	2481	95	196	1.17	1.09
14.33	747.67	2836	2668	95	196	1.17	1.10
15.33	746.67	3066	2854	95	196	1.18	1.11

Conclusions
For levee stretch: 5+00UE to 66+00
For pipe diam (in): 24
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 14.33 feet and less at the landward toe

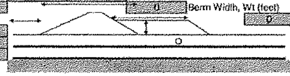
Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	Ll	L2	Lc	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
5+00UE	300	300	22	22	22	70	15.5	0.84	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.000
60+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr Levee Width, L2

200 | 100

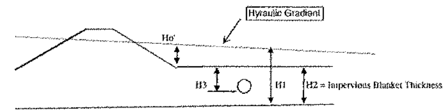
Levee Elev. 777.5 — 0 Berm Width, Wt (feet)

0 Berm Height, t (feet)



Ground Elev.	762	
Blanket Base Elev.	740	
Bedrock Elev.	670	

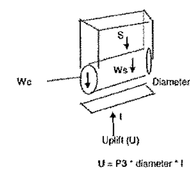
10	Pipe Depth (ft)
752	Pipe Elev.
24	Pipe Diameter (in)
ML	Soil Type
115	Soil Unit Weight, γsoil (pcf)
10	Pipe Depth (Check)



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 24 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	752	740	762	10	22	32.8	930.8	95	196	1840	1862	0	1.14	1.04
25	10.42	752	740	762	10	22	32.4	919.7	95	196	1840	1839	0	1.16	1.05
50	10.05	752	740	762	10	22	32.0	909.0	95	196	1840	1818	0	1.17	1.06
100	9.34	752	740	762	10	22	31.3	888.8	95	196	1840	1778	0	1.20	1.09
125	9.00	752	740	762	10	22	31.0	879.2	95	196	1840	1758	0	1.21	1.10
150	8.67	752	740	762	10	22	30.7	870.0	95	196	1840	1740	0	1.22	1.11
200	8.06	752	740	762	10	22	30.1	852.6	95	196	1840	1705	0	1.25	1.13
300	6.96	752	740	762	10	22	29.0	821.3	95	196	1840	1643	0	1.30	1.18
400	6.00	752	740	762	10	22	28.0	794.3	95	196	1840	1589	0	1.34	1.22
500	5.18	752	740	762	10	22	27.2	771.0	95	196	1840	1542	0	1.38	1.25

Ws Calc 24 -in steel 1ff
95
Wc Calc
196

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 94.62 lb per ft (24-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((12/12)^2) *1*62.4 = 196 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l

Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

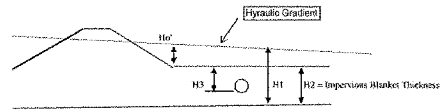
	Ground Elev 762		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 95		Wc = 196 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	762	0		95	196		
10	752	1840	1862	95	196	1.14	1.04
11	751	2070	2048	95	196	1.16	1.06
12	750	2300	2234	95	196	1.16	1.07
13	749	2530	2420	95	196	1.17	1.08
14	748	2760	2606	95	196	1.17	1.10
15	747	2990	2792	95	196	1.18	1.10
16	746	3220	2978	95	196	1.18	1.11
17	745	3450	3165	95	196	1.18	1.12
18	744	3680	3351	95	196	1.19	1.13
19	743	3910	3537	95	196	1.19	1.13
20	742	4140	3723	95	196	1.19	1.14
21	741	4370	3909	95	196	1.19	1.14
22	740	4600	4095	95	196	1.19	1.15

Conclusions
For levee stretch: 5+00UE to 60+00
For pipe diam (in): 24
There is uplift concern at a depth of 10 feet and less within 100 feet landward of the levee
There is uplift concern at a depth of 14 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Lc	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
5+00UE	300	300	22	22	22	70	15.5	0.84	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.000
66+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr Levee Width, L2

Levee Elev.	777.5	
Lr	200	
Levee Width, L2	100	
Berm Width, Wt (feet)	0	
Berm Height, t (feet)	0	
Ground Elev.	762	
Blanket Base Elev.	740	
Bedrock Elev.	670	
Pipe Depth (ft)	15	
Pipe Elev.	747	
Pipe Diameter (in)	24	
Soil Type	ML	
Soil Unit Weight, γsoil (pcf)	115	
Pipe Depth (Check)	15	

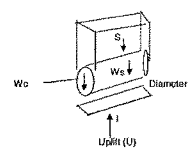


d=Landside No Berm Calcs

Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 24-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	747	740	762	15	22	32.8	1396.1	95	196	2990	2792	0	1.18	1.10
25	10.42	747	740	762	15	22	32.4	1379.5	95	196	2990	2759	0	1.19	1.12
50	10.05	747	740	762	15	22	32.0	1363.5	95	196	2990	2727	0	1.20	1.13
100	9.34	747	740	762	15	22	31.3	1333.2	95	196	2990	2666	0	1.23	1.16
125	9.00	747	740	762	15	22	31.0	1318.8	95	196	2990	2638	0	1.24	1.17
150	8.67	747	740	762	15	22	30.7	1305.0	95	196	2990	2610	0	1.26	1.18
200	8.06	747	740	762	15	22	30.1	1278.6	95	196	2990	2558	0	1.28	1.21
300	6.96	747	740	762	15	22	29.0	1231.9	95	196	2990	2464	0	1.33	1.25
400	6.00	747	740	762	15	22	28.0	1191.5	95	196	2990	2383	0	1.38	1.29
500	5.16	747	740	762	15	22	27.2	1156.5	95	196	2990	2313	0	1.42	1.33

Ws Calc 24-in steel 1ft
95
Wc Calc
196

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 94.62 lb per ft (24-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((12/12)^2) *1*62.4 = 196 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

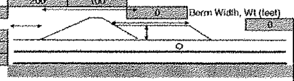
	Ground Elev 762		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 95		Wc = 196 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	762	0		95	196		
15	747	2990	2792	95	196	1.18	1.10
16	746	3220	2978	95	196	1.18	1.11
17	745	3450	3165	95	196	1.19	1.12
18	744	3680	3351	95	196	1.19	1.13
19	743	3910	3537	95	196	1.19	1.13
20	742	4140	3723	95	196	1.19	1.14
21	741	4370	3909	95	196	1.19	1.14
22	740	4600	4095	95	196	1.19	1.15
23	739	4830	4281	95	196	1.20	1.15
24	738	5060	4468	95	196	1.20	1.15
25	737	5290	4654	95	196	1.20	1.16
26	736	5520	4840	95	196	1.20	1.16
27	735	5750	5026	95	196	1.20	1.16

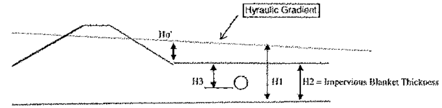
Conclusions
For levee stretch: 5+00UE to 66+00
For pipe diam (in): 24
There is no uplift concern at a depth of 15 feet or deeper within 500 feet landward of the levee

Armourdale Area	K-r	K-L	DhL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Lc	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
5+00UE	300	300	22	22	22	70	15.5	0.84	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.000
66+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.																							
500-yr + 3ft																								

Lr Levee Width, L2

Lr	200
Levee Width, L2	100
Levee Elev.	777.5
Berm Width, Wt (feet)	0
Berm Height, t (feet)	0
Ground Elev.	762
Blanket Base Elev.	740
Bedrock Elev.	670
Pipe Depth (ft)	6
Pipe Elev.	756
Pipe Diameter (in)	48
Soil Type	ML
Soil Unit Weight, γsoil (pcf)	115
Pipe Depth (Check)	6

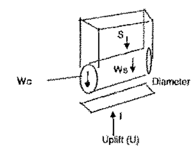




d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 48-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	756	740	762	6	22	32.8	558.5	191	784	920	2234	0	0.85	0.50
25	10.42	756	740	762	6	22	32.4	551.8	191	784	920	2207	0	0.86	0.50
50	10.05	756	740	762	6	22	32.0	545.4	191	784	920	2182	0	0.87	0.51
100	9.34	756	740	762	6	22	31.3	533.3	191	784	920	2133	0	0.89	0.52
125	9.00	756	740	762	6	22	31.0	527.5	191	784	920	2110	0	0.90	0.53
150	8.67	756	740	762	6	22	30.7	522.0	191	784	920	2088	0	0.91	0.53
200	8.06	756	740	762	6	22	30.1	511.5	191	784	920	2046	0	0.93	0.54
300	6.96	756	740	762	6	22	29.0	492.8	191	784	920	1971	0	0.96	0.56
400	6.00	756	740	762	6	22	28.0	476.6	191	784	920	1906	0	0.99	0.58
500	5.18	756	740	762	6	22	27.2	462.6	191	784	920	1850	0	1.02	0.60

Ws Calc 48-in steel 1ft
191
Wc Calc 784

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 190.70 lb per ft (48-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 780 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 762		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 191		Wc = 784 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	762	0		191	784		
6	756	920	2234	191	784	0.85	0.50
10	752	2760	3723	191	784	1.00	0.79
14	748	4600	5212	191	784	1.07	0.92
18	744	6440	6701	191	784	1.11	0.99
22	740	8280	8191	191	784	1.13	1.03
26	736	10120	9680	191	784	1.15	1.07
30	732	11960	11169	191	784	1.16	1.09
34	728	13800	12658	191	784	1.17	1.11
38	724	15640	14148	191	784	1.17	1.12

Conclusions
For levee stretch: 5+00UE to 66+00
For pipe diam (in): 48
There is uplift concern at a depth of 6 feet and less within 500 feet landward of the levee
There is uplift concern at a depth of 30 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Lc	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
5+00UE	300	300	22	22	22	70	15.5	0.84	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.000
66+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.																							
500-yr + 3ft																								

Lr Levee Width, L2

200 | 100

Levee Elev. 777.5

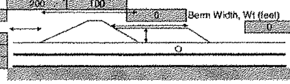
0 Berm Width, Wt (feet)

0 Berm Height, t (feet)

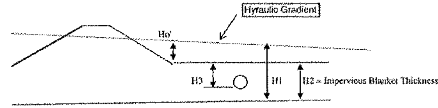
Ground Elev. 762

Blanket Base Elev. 740

Bedrock Elev. 670



15	Pipe Depth (ft)
747	Pipe Elev.
48	Pipe Diameter (in)
ML	Soil Type
115	Soil Unit Weight, γsoil (pcf)
15	Pipe Depth (Check)



No Berm Calcs

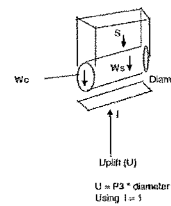
d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 48 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	747	740	762	15	22	32.8	1396.1	191	784	5060	5585	0	1.08	0.94
25	10.42	747	740	762	15	22	32.4	1379.5	191	784	5060	5518	0	1.09	0.95
50	10.05	747	740	762	15	22	32.0	1363.5	191	784	5060	5454	0	1.11	0.96
100	9.34	747	740	762	15	22	31.3	1333.2	191	784	5060	5333	0	1.13	0.98
125	9.00	747	740	762	15	22	31.0	1318.8	191	784	5060	5275	0	1.14	1.00
150	8.67	747	740	762	15	22	30.7	1305.0	191	784	5060	5220	0	1.16	1.01
200	8.06	747	740	762	15	22	30.1	1278.8	191	784	5060	5115	0	1.18	1.03
300	6.96	747	740	762	15	22	29.0	1231.9	191	784	5060	4928	0	1.22	1.07
400	6.00	747	740	762	15	22	28.0	1191.5	191	784	5060	4766	0	1.27	1.10
500	5.18	747	740	762	15	22	27.2	1156.5	191	784	5060	4626	0	1.30	1.14

Ws Calc
48 -In steel 1ft
191
Wc Calc
784

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 190.70 lb per ft (48-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 780 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 762		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 191		Wc = 784 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	762	0		191	784		
15	747	5060	5585	191	784	1.08	0.94
17	745	5980	6329	191	784	1.10	0.97
19	743	6900	7074	191	784	1.11	1.00
21	741	7820	7818	191	784	1.12	1.02
23	739	8740	8563	191	784	1.13	1.04
25	737	9660	9308	191	784	1.14	1.06
27	735	10580	10052	191	784	1.15	1.07
29	733	11500	10797	191	784	1.16	1.08
31	731	12420	11541	191	784	1.16	1.09
33	729	13340	12286	191	784	1.17	1.10
35	727	14260	13031	191	784	1.17	1.11



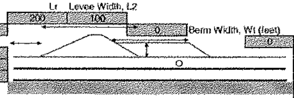
Conclusions

For levee stretch: 5+00UE to 66+00
For pipe diam (in): 48
There is uplift concern at a depth of 15 feet and less within 300 feet landward of the levee
There is uplift concern at a depth of 31 feet and less at the landward toe

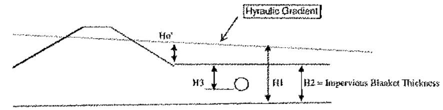
Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
5+00UE	300	300	22	22	22	70	15.5	0.84	200	0	1.71	194	100	680	974	974	10.82	10.82	10.82	0.49	680	680	0	0.000
66+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.																							
500-yr + 3ft																								

Lr Levee Width, L2

Levee Elev.	777.5
Ground Elev.	762
Blanket Base Elev.	740
Bedrock Elev.	670



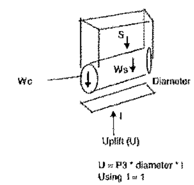
25	Pipe Depth (ft)
737	Pipe Elev.
48	Pipe Diameter (in)
ML	Soil Type
115	Soil Unit Weight, γsoil (pcf)
25	Pipe Depth (Check)



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 48 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.82	737	740	762	25	22	32.8	2326.9	191	784	9660	9308	0	1.14	1.06
25	10.42	737	740	762	25	22	32.4	2299.2	191	784	9660	9197	0	1.16	1.07
50	10.05	737	740	762	25	22	32.0	2272.5	191	784	9660	9090	0	1.17	1.08
100	9.34	737	740	762	25	22	31.3	2222.0	191	784	9660	8888	0	1.20	1.11
125	9.00	737	740	762	25	22	31.0	2198.1	191	784	9660	8792	0	1.21	1.12
150	8.67	737	740	762	25	22	30.7	2175.0	191	784	9660	8700	0	1.22	1.13
200	8.06	737	740	762	25	22	30.1	2131.4	191	784	9660	8526	0	1.25	1.16
300	6.96	737	740	762	25	22	29.0	2053.2	191	784	9660	8213	0	1.29	1.20
400	6.00	737	740	762	25	22	28.0	1985.8	191	784	9660	7943	0	1.34	1.24
500	5.18	737	740	762	25	22	27.2	1927.5	191	784	9660	7710	0	1.38	1.28

Ws Calc 48 -in steel 1ft
191
Wc Calc 784

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 190.70 lb per ft (48-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 780 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

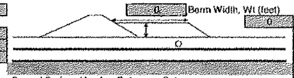
	Ground Elev 762		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 191		Wc = 784 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	762	0		191	784		
25	737	9660	9308	191	784	1.14	1.06
26	736	10120	9680	191	784	1.15	1.07
27	735	10580	10052	191	784	1.15	1.07
28	734	11040	10425	191	784	1.15	1.08
29	733	11500	10797	191	784	1.16	1.08
30	732	11960	11169	191	784	1.16	1.09
31	731	12420	11541	191	784	1.16	1.09
32	730	12880	11914	191	784	1.16	1.10
33	729	13340	12286	191	784	1.17	1.10

Conclusions

For levee stretch: 5+00UE to 66+00
For pipe diam (in): 48
There is uplift concern at a depth of 25 feet and less within 50 feet landward of the levee
There is uplift concern at a depth of 32 feet and less at the landward toe

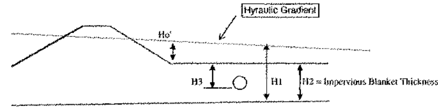
Armourdale Area
L2
Stationing Range
68+00
79+00
500-yr + 3ft

Levee Elev.	777
Ground Elev.	760
Blanket Base Elev.	748
Bedrock Elev.	671



0	Berm Width, Wt (feet)
0	Berm Height, t (feet)
3.3	Pipe Dept (ft)
756.7	Pipe Elev.
6	Pipe Diameter (in)
115	Soil Unit Weight, γsoil (pcf)
3.3	Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum



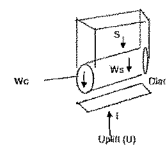
From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

Dist from toe FT	Ho FT
0	7
500	4

d=Landside Distance from Toe Feet	No Berm Calcs With WELLS Excess Head Feet, Ho'	Invert Elev of pipe 6-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	756.7	748	760	3.3	12	19.0	326.0	19	12	161	163	0	1.18	1.10
25	6.9	756.7	748	760	3.3	12	18.9	323.5	19	12	161	162	0	1.19	1.11
50	6.7	756.7	748	760	3.3	12	18.7	320.9	19	12	161	160	0	1.20	1.12
100	6.4	756.7	748	760	3.3	12	18.4	315.7	19	12	161	158	0	1.22	1.14
125	6.3	756.7	748	760	3.3	12	18.3	313.2	19	12	161	157	0	1.23	1.15
150	6.1	756.7	748	760	3.3	12	18.1	310.6	19	12	161	155	0	1.24	1.16
200	5.8	756.7	748	760	3.3	12	17.8	305.4	19	12	161	153	0	1.26	1.18
300	5.2	756.7	748	760	3.3	12	17.2	295.2	19	12	161	148	0	1.30	1.22
400	4.6	756.7	748	760	3.3	12	16.6	284.9	19	12	161	142	0	1.35	1.26
500	4.0	756.7	748	760	3.3	12	16.0	274.6	19	12	161	137	0	1.40	1.31

Ws Calc 6-in steel 1ft
19
Wc Calc
12

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 18.97 lb per ft (6-inch Diameter Steel Pipe, .280" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((3/12)^2) *1*62.4 = 12 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * 1
Using 1 = 1

Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 6
There is no uplift concern at a depth of 3.33 feet or deeper within 500 feet landward of the levee

Armourdale Area
L2
Stationing Range
66+00
70+00
500-yr + 3ft

Levee Elev. 777

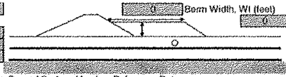
0 Berm Width, Wl (feet)

0 Berm Height, t (feet)

Ground Elev. 760

Blanket Base Elev. 748

Bedrock Elev. 671



3.3 Pipe Depth (ft)

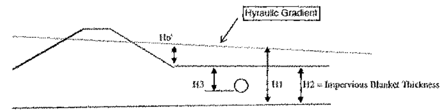
756.7 Pipe Elev.

12 Pipe Diameter (in)

115 Soil Unit Weight, γsoil (pcf)

3.3 Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

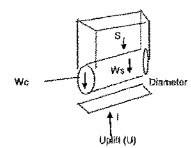
Dist from toe FT	Ho FT
0	7
500	4

d=Landside — No Berm Calcs With WELLS

Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 12-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	756.7	748	760	3.3	12	19.0	326.0	50	49	264	326	0	1.11	0.96
25	6.9	756.7	748	760	3.3	12	18.9	323.5	50	49	264	323	0	1.12	0.97
50	6.7	756.7	748	760	3.3	12	18.7	320.9	50	49	264	321	0	1.13	0.98
100	6.4	756.7	748	760	3.3	12	18.4	315.7	50	49	264	316	0	1.15	0.99
125	6.3	756.7	748	760	3.3	12	18.3	313.2	50	49	264	313	0	1.16	1.00
150	6.1	756.7	748	760	3.3	12	18.1	310.6	50	49	264	311	0	1.17	1.01
200	5.8	756.7	748	760	3.3	12	17.8	305.4	50	49	264	305	0	1.19	1.03
300	5.2	756.7	748	760	3.3	12	17.2	295.2	50	49	264	295	0	1.23	1.06
400	4.6	756.7	748	760	3.3	12	16.6	284.9	50	49	264	285	0	1.27	1.10
500	4.0	756.7	748	760	3.3	12	16.0	274.6	50	49	264	275	0	1.32	1.14

Ws Calc
12-in steel 1lf
50
Wc Calc
49

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 50 lb per ft (12-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * l = 3.1416*((8/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

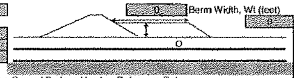


U = P3 * diameter * l
Using l = 1

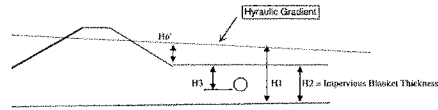
Conclusions
For levee stretch: 66+00 to 70+00
For pipe diam (in): 12
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 300 feet landward of the levee

Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev.	777	
	0	Berm Width, Wt (feet)
	0	Berm Height, t (feet)
Ground Elev.	760	
Blanket Base Elev.	748	
Bedrock Elev.	671	
	10	Pipe Depth (ft)
	750	Pipe Elev.
	12	Pipe Diameter (in)
	115	Soil Unit Weight, γsoil (pcf)
	10	Pipe Depth (Check)



Ground Elev = Landside Average Ground Surface Used as Reference Datum



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

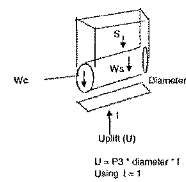
Dist from toe FT	Ho FT
0	7
500	4

No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 12 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	750	748	760	10	12	19.0	988.0	50	49	1035	988	0	1.15	1.10
25	6.9	750	748	760	10	12	18.9	980.2	50	49	1035	980	0	1.16	1.11
50	6.7	750	748	760	10	12	18.7	972.4	50	49	1035	972	0	1.17	1.12
100	6.4	750	748	760	10	12	18.4	956.8	50	49	1035	957	0	1.18	1.13
125	6.3	750	748	760	10	12	18.3	949.0	50	49	1035	949	0	1.19	1.14
150	6.1	750	748	760	10	12	18.1	941.2	50	49	1035	941	0	1.20	1.15
200	5.8	750	748	760	10	12	17.8	925.6	50	49	1035	926	0	1.22	1.17
300	5.2	750	748	760	10	12	17.2	894.4	50	49	1035	894	0	1.27	1.21
400	4.6	750	748	760	10	12	16.6	863.2	50	49	1035	863	0	1.31	1.26
500	4.0	750	748	760	10	12	16.0	832.0	50	49	1035	832	0	1.36	1.30

Ws Calc 12 -in steel 1ft
50
Wc Calc
49

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 50 lb per ft (12-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((6/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

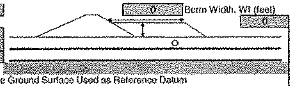


Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 12
There is uplift concern at a depth of 40 inches (3.3 feet) and less at the toe of the levee

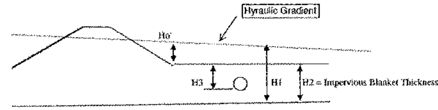
Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev. 777
Ground Elev. 760
Blanket Base Elev. 748
Bedrock Elev. 671

0 Berm Width, Wt (feet)
0 Berm Height, t (feet)
3.33 Pipe Depth (ft)
756.67 Pipe Elev.
16 Pipe Diameter (in)
116 Soil Unit Weight, γsoil (pcf)
3.33 Pipe Depth (Check)



Ground Elev = Landside Average Ground Surface Used as Reference Datum



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

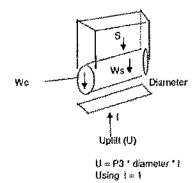
Dist from toe FT	Ho FT
0	7
500	4

d=Landside — No Berm Calcs With WELLS

Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 16-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	756.67	748	760	3.33	12	19.0	329.0	63	87	306	439	0	1.04	0.84
25	6.9	756.67	748	760	3.33	12	18.9	326.4	63	87	306	435	0	1.05	0.85
50	6.7	756.67	748	760	3.33	12	18.7	323.8	63	87	306	432	0	1.06	0.86
100	6.4	756.67	748	760	3.33	12	18.4	318.6	63	87	306	425	0	1.07	0.87
125	6.3	756.67	748	760	3.33	12	18.3	316.0	63	87	306	421	0	1.08	0.88
150	6.1	756.67	748	760	3.33	12	18.1	313.4	63	87	306	416	0	1.09	0.88
200	5.8	756.67	748	760	3.33	12	17.8	308.2	63	87	306	411	0	1.11	0.90
300	5.2	756.67	748	760	3.33	12	17.2	297.8	63	87	306	397	0	1.15	0.93
400	4.6	756.67	748	760	3.33	12	16.6	287.4	63	87	306	383	0	1.19	0.96
500	4.0	756.67	748	760	3.33	12	16.0	277.1	63	87	306	369	0	1.24	1.00

Ws Calc 16-in steel 1lf
63
Wc Calc
87

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 62.58 lb per ft (16-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((8/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

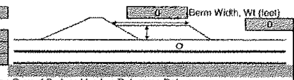


U = P3 * diameter * 1
Using 1 = 1

Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 16
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet of the levee

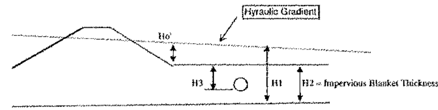
Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev.	777
Ground Elev.	760
Blanket Base Elev.	748
Bedrock Elev.	671



0	Berm Width, Wt (feet)
0	Berm Height, t (feet)
10	Pipe Depth (ft)
750	Pipe Elev.
16	Pipe Diameter (in)
115	Soil Unit Weight, γsoil (pcf)
10	Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

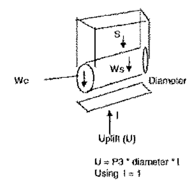
Dist from toe FT	Ho FT
0	7
500	4

No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 16-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	750	748	760	10	12	19.0	988.0	63	87	1329	1317	0	1.12	1.06
25	6.9	750	748	760	10	12	18.9	980.2	63	87	1329	1307	0	1.13	1.07
50	6.7	750	748	760	10	12	18.7	972.4	63	87	1329	1297	0	1.14	1.07
100	6.4	750	748	760	10	12	18.4	956.8	63	87	1329	1276	0	1.16	1.09
125	6.3	750	748	760	10	12	18.3	949.0	63	87	1329	1265	0	1.17	1.10
150	6.1	750	748	760	10	12	18.1	941.2	63	87	1329	1255	0	1.18	1.11
200	5.8	750	748	760	10	12	17.8	925.6	63	87	1329	1234	0	1.20	1.13
300	5.2	750	748	760	10	12	17.2	894.4	63	87	1329	1193	0	1.24	1.17
400	4.6	750	748	760	10	12	16.6	863.2	63	87	1329	1151	0	1.29	1.21
500	4.0	750	748	760	10	12	16.0	832.0	63	87	1329	1109	0	1.33	1.25

Ws Calc 16-in steel 1lf
63
Wc Calc 87

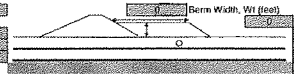
See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 62.58 lb per ft (16-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((8/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 16
There is uplift concern at a depth of 10 feet and less within 100 feet from the toe of the levee

Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev. 777

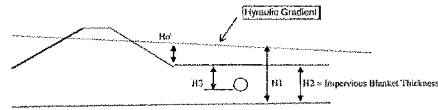


Ground Elev.	760	
Blanket Base Elev.	748	
Bedrock Elev.	671	
0	Berm Width, Wf (feet)	
0	Berm Height, t (feet)	
15	Pipe Depth (ft)	
745	Pipe Elev.	
16	Pipe Diameter (in)	
115	Soil Unit Weight, γsoil (pcf)	
15	Pipe Depth (Check)	

Ground Elev = Landside Average Ground Surface Used as Reference Datum

From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

Dist from toe FT	Ho FT
0	7
500	4

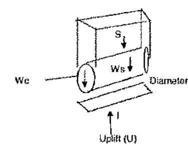


No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 16 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	745	748	760	15	12	19.0	1482.0	63	87	2096	1976	0	1.14	1.09
25	6.9	745	748	760	15	12	18.9	1470.3	63	87	2096	1960	0	1.16	1.10
50	6.7	745	748	760	15	12	18.7	1458.6	63	87	2096	1945	0	1.15	1.11
100	6.4	745	748	760	15	12	18.4	1435.2	63	87	2096	1914	0	1.17	1.13
125	6.3	745	748	760	15	12	18.3	1423.5	63	87	2096	1898	0	1.18	1.14
150	6.1	745	748	760	15	12	18.1	1411.8	63	87	2096	1882	0	1.19	1.15
200	5.8	745	748	760	15	12	17.8	1388.4	63	87	2096	1851	0	1.21	1.17
300	5.2	745	748	760	15	12	17.2	1341.6	63	87	2096	1789	0	1.26	1.21
400	4.6	745	748	760	15	12	16.6	1294.8	63	87	2096	1726	0	1.30	1.25
500	4.0	745	748	760	15	12	16.0	1248.0	63	87	2096	1664	0	1.35	1.30

Ws Calc 16 -in steel 1lf
63
Wc Calc
87

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 62.58 lb per ft (16-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((8/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 16
There is uplift concern at a depth of 15 feet and less just at the toe of the levee

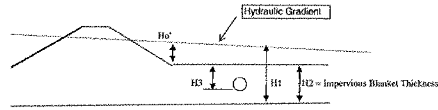
Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev. 777

Ground Elev. 760
Blanket Base Elev. 748
Bedrock Elev. 671

Ground Elev = Landside Average Ground Surface Used as Reference Datum

115 Soil Unit Weight, γsoil (pcf)
3.3 Pipe Depth (Check)



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

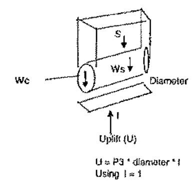
Dist from toe FT	Ho FT
0	7
500	4

d=Landside — No Berm Calcs With WELLS

Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 24 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	756.7	748	760	3.3	12	19.0	326.0	95	196	299	652	0	0.90	0.60
25	6.9	756.7	748	760	3.3	12	18.9	323.5	95	196	299	647	0	0.91	0.61
50	6.7	756.7	748	760	3.3	12	18.7	320.9	95	196	299	642	0	0.92	0.61
100	6.4	756.7	748	760	3.3	12	18.4	315.7	95	196	299	631	0	0.93	0.62
125	6.3	756.7	748	760	3.3	12	18.3	313.2	95	196	299	626	0	0.94	0.63
150	6.1	756.7	748	760	3.3	12	18.1	310.6	95	196	299	621	0	0.95	0.63
200	5.8	756.7	748	760	3.3	12	17.8	305.4	95	196	299	611	0	0.97	0.64
300	5.2	756.7	748	760	3.3	12	17.2	295.2	95	196	299	590	0	1.00	0.67
400	4.6	756.7	748	760	3.3	12	16.6	284.9	95	196	299	570	0	1.04	0.69
500	4.0	756.7	748	760	3.3	12	16.0	274.6	95	196	299	549	0	1.07	0.72

Ws Calc
24 -in steel 1lf
95
Wc Calc
196

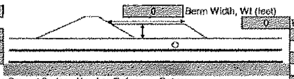
See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 94.62 lb per ft (24-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((12/12)^2) *1*62.4 = 196 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 24
There is uplift concern at a depth of 3.33 feet or less within 500 feet landward of the levee

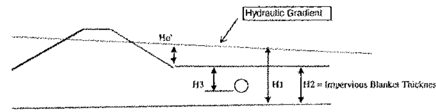
Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev.	777
Ground Elev.	760
Blanket Base Elev.	748
Bedrock Elev.	671



0	Berm Width, Wt (feet)
0	Berm Height, t (feet)
10	Pipe Depth (ft)
750	Pipe Elev.
24	Pipe Diameter (in)
115	Soil Unit Weight, γsoil (pcf)
10	Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

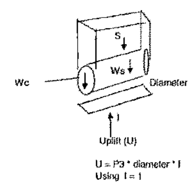
Dist from toe FT	Ho FT
0	7
500	4

No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 24 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	750	748	760	10	12	19.0	988.0	95	196	1840	1976	0	1.08	0.98
25	6.9	750	748	760	10	12	18.9	980.2	95	196	1840	1960	0	1.09	0.99
50	6.7	750	748	760	10	12	18.7	972.4	95	196	1840	1945	0	1.10	0.99
100	6.4	750	748	760	10	12	18.4	956.8	95	196	1840	1914	0	1.11	1.01
125	6.3	750	748	760	10	12	18.3	949.0	95	196	1840	1898	0	1.12	1.02
150	6.1	750	748	760	10	12	18.1	941.2	95	196	1840	1882	0	1.13	1.03
200	5.8	750	748	760	10	12	17.8	925.6	95	196	1840	1851	0	1.15	1.05
300	5.2	750	748	760	10	12	17.2	894.4	95	196	1840	1789	0	1.19	1.08
400	4.6	750	748	760	10	12	16.6	863.2	95	196	1840	1726	0	1.23	1.12
500	4.0	750	748	760	10	12	16.0	832.0	95	196	1840	1664	0	1.28	1.16

Ws Calc
24 -in steel 1ft
95
Wc Calc
196

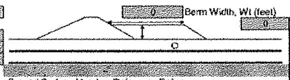
See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 94.62 lb per ft (24-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((12/12)^2) *1*62.4 = 196 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 24
There is uplift concern at a depth of 10 feet or less within 300 feet landward of the levee

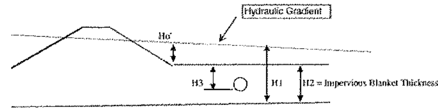
Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev. 777
Ground Elev. 760
Blanket Base Elev. 748
Bedrock Elev. 671



0 Berm Width, Wt (feet)
0 Berm Height, t (feet)
15 Pipe Depth (ft)
745 Pipe Elev.
24 Pipe Diameter (in)
115 Soil Unit Weight, γsoil (pcf)
15 Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum



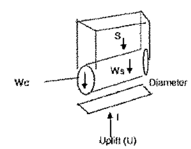
From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

Dist from toe FT	Ho FT
0	7
500	4

d=Landside	No Berm Calcs With WELLS														
Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 24 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	745	748	760	15	12	19.0	1482.0	95	196	2990	2964	0	1.11	1.04
25	6.9	745	748	760	15	12	18.9	1470.3	95	196	2990	2941	0	1.12	1.05
50	6.7	745	748	760	15	12	18.7	1458.6	95	196	2990	2917	0	1.12	1.06
100	6.4	745	748	760	15	12	18.4	1435.2	95	196	2990	2870	0	1.14	1.07
125	6.3	745	748	760	15	12	18.3	1423.5	95	196	2990	2847	0	1.15	1.08
150	6.1	745	748	760	15	12	18.1	1411.8	95	196	2990	2824	0	1.16	1.09
200	5.8	745	748	760	15	12	17.8	1388.4	95	196	2990	2777	0	1.18	1.11
300	5.2	745	748	760	15	12	17.2	1341.6	95	196	2990	2683	0	1.22	1.15
400	4.6	745	748	760	15	12	16.6	1294.8	95	196	2990	2590	0	1.27	1.19
500	4.0	745	748	760	15	12	16.0	1248.0	95	196	2990	2496	0	1.31	1.24

Ws Calc
24 -in steel 1lf
95
Wc Calc
196

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 94.62 lb per ft (24-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((12/12)^2) *1*62.4 = 196 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

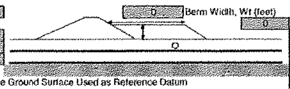


U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 24
There is uplift concern at a depth of 15 feet or less within 150 feet landward of the levee

Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev.	777
Ground Elev.	760
Blanket Base Elev.	748
Bedrock Elev.	671

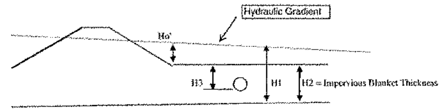


20	Pipe Depth (ft)
740	Pipe Elev.
24	Pipe Diameter (in)
115	Soil Unit Weight, γsoil (pcf)
20	Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum

From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

Dist from toe FT	Ho FT
0	7
500	4

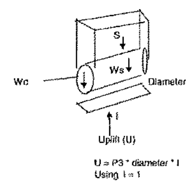


No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 24 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	740	748	760	20	12	19.0	1976.0	95	196	4140	3952	0	1.12	1.07
25	6.9	740	748	760	20	12	18.9	1960.4	95	196	4140	3921	0	1.13	1.08
50	6.7	740	748	760	20	12	18.7	1944.8	95	196	4140	3890	0	1.14	1.09
100	6.4	740	748	760	20	12	18.4	1913.6	95	196	4140	3827	0	1.16	1.11
125	6.3	740	748	760	20	12	18.3	1898.0	95	196	4140	3796	0	1.17	1.12
150	6.1	740	748	760	20	12	18.1	1882.4	95	196	4140	3765	0	1.18	1.12
200	5.8	740	748	760	20	12	17.8	1851.2	95	196	4140	3702	0	1.20	1.14
300	5.2	740	748	760	20	12	17.2	1788.8	95	196	4140	3578	0	1.24	1.18
400	4.6	740	748	760	20	12	16.6	1726.4	95	196	4140	3453	0	1.28	1.23
500	4.0	740	748	760	20	12	16.0	1664.0	95	196	4140	3328	0	1.33	1.27

Ws Calc
24 -in steel 1ft
95
Wc Calc
196

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 94.62 lb per ft (24-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((12/12)^2) *1*62.4 = 196 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 24
There is uplift concern at a depth of 15 feet or less within 50 feet landward of the levee

Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev. 777

Ground Elev. 760

Blanket Base Elev. 748

Bedrock Elev. 671

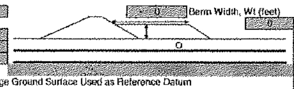
0 Berm Width, Wt (feet)

0 Berm Height, t (feet)

25 Pipe Depth (ft)

735 Pipe Elev.

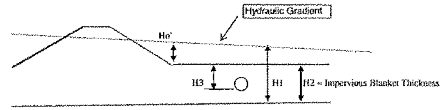
24 Pipe Diameter (in)



Ground Elev = Landside Average Ground Surface Used as Reference Datum

115 Soil Unit Weight, γsoil (pcf)

25 Pipe Depth (Check)



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

Dist from toe FT	Ho FT
0	7
500	4

No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 24 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	735	748	760	25	12	19.0	2470.0	95	196	5290	4940	0	1.13	1.09
25	6.9	735	748	760	25	12	18.9	2450.5	95	196	5290	4901	0	1.14	1.10
50	6.7	735	748	760	25	12	18.7	2431.0	95	196	5290	4862	0	1.15	1.11
100	6.4	735	748	760	25	12	18.4	2392.0	95	196	5290	4784	0	1.17	1.13
125	6.3	735	748	760	25	12	18.3	2372.5	95	196	5290	4745	0	1.18	1.13
150	6.1	735	748	760	25	12	18.1	2353.0	95	196	5290	4706	0	1.19	1.14
200	5.8	735	748	760	25	12	17.8	2314.0	95	196	5290	4628	0	1.21	1.16
300	5.2	735	748	760	25	12	17.2	2236.0	95	196	5290	4472	0	1.25	1.20
400	4.6	735	748	760	25	12	16.6	2158.0	95	196	5290	4316	0	1.29	1.25
500	4.0	735	748	760	25	12	16.0	2080.0	95	196	5290	4160	0	1.34	1.29

Ws Calc 24 -in steel 1lf
95
Wc Calc
196

See Sample Calculations for list of abbreviations and sample calculations

Ws = weight of structure per foot of length = 94.62 lb per ft (24-inch Diameter Steel Pipe, .375" thick)

Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((12/12)^2) *1*62.4 = 196 plf

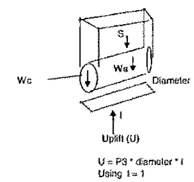
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil

P3 = H3*(H1/H2)*γwater

U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3

Wg = weight of surcharge water above top surface of structure control by gravity flow

SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Conclusions

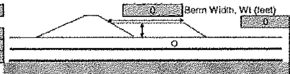
For levee stretch: 66+00 to 79+00

For pipe diam (in): 24

There is uplift concern at a depth of 15 feet or less within 25 feet landward of the levee

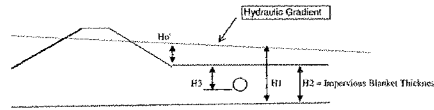
Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev.	777	
Ground Elev.	760	
Blanket Base Elev.	748	
Bedrock Elev.	671	



0	Berm Width, Wt (feet)
0	Berm Height, t (feet)
30	Pipe Depth (ft)
730	Pipe Elev.
24	Pipe Diameter (in)
115	Soil Unit Weight, γsoil (pcf)
30	Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

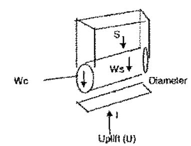
Dist from toe FT	Ho FT
0	7
500	4

No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 24 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	730	748	760	30	12	19.0	2964.0	95	196	6440	5928	0	1.14	1.10
25	6.9	730	748	760	30	12	18.9	2940.6	95	196	6440	5881	0	1.14	1.11
50	6.7	730	748	760	30	12	18.7	2917.2	95	196	6440	5834	0	1.15	1.12
100	6.4	730	748	760	30	12	18.4	2870.4	95	196	6440	5741	0	1.17	1.14
125	6.3	730	748	760	30	12	18.3	2847.0	95	196	6440	5694	0	1.18	1.15
150	6.1	730	748	760	30	12	18.1	2823.6	95	196	6440	5647	0	1.19	1.16
200	5.8	730	748	760	30	12	17.8	2776.8	95	196	6440	5554	0	1.21	1.18
300	5.2	730	748	760	30	12	17.2	2683.2	95	196	6440	5366	0	1.25	1.22
400	4.6	730	748	760	30	12	16.6	2589.6	95	196	6440	5179	0	1.30	1.26
500	4.0	730	748	760	30	12	16.0	2496.0	95	196	6440	4992	0	1.35	1.31

Ws Calc 24 -in steel 1ft
95
Wc Calc 196

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 94.62 lb per ft (24-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((12/12)^2) *1*62.4 = 196 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



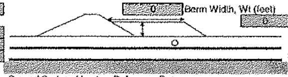
U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 24
There is no uplift concern at a depth of 30 feet or less within 500 feet landward of the levee

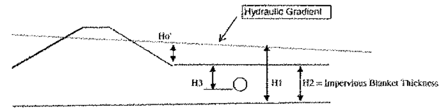
Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev. 777 | 0 Berm Width, Wt (feet)
0 Berm Height, t (feet)

Ground Elev. 760 | 15 Pipe Dept (ft)
Blanket Base Elev. 748 | 745 Pipe Elev.
Bedrock Elev. 673 | 48 Pipe Diameter (in)



Ground Elev = Landside Average Ground Surface Used as Reference Datum | 115 Soil Unit Weight, γsoil (pcf)
15 Pipe Depth (Check)



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

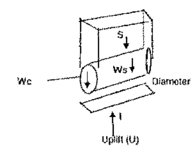
Dist from toe FT	Ho FT
0	7
500	4

No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 48 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	745	748	760	15	12	19.0	1482.0	191	784	5060	5928	0	1.02	0.89
25	6.9	745	748	760	15	12	18.9	1470.3	191	784	5060	5881	0	1.03	0.89
50	6.7	745	748	760	15	12	18.7	1458.6	191	784	5060	5834	0	1.03	0.90
100	6.4	745	748	760	15	12	18.4	1435.2	191	784	5060	5741	0	1.05	0.91
125	6.3	745	748	760	15	12	18.3	1423.5	191	784	5060	5694	0	1.06	0.92
150	6.1	745	748	760	15	12	18.1	1411.8	191	784	5060	5647	0	1.07	0.93
200	5.8	745	748	760	15	12	17.8	1388.4	191	784	5060	5554	0	1.09	0.95
300	5.2	745	748	760	15	12	17.2	1341.6	191	784	5060	5366	0	1.12	0.98
400	4.6	745	748	760	15	12	16.6	1294.8	191	784	5060	5179	0	1.17	1.01
500	4.0	745	748	760	15	12	16.0	1248.0	191	784	5060	4992	0	1.21	1.05

Ws Calc 48 -in steel tlf
191
Wc Calc 784

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 190.70 lb per ft (48-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 780 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

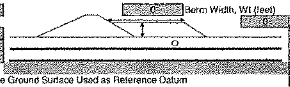


U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 48
There is uplift concern at a depth of 15 feet or less within 500 feet landward of the levee

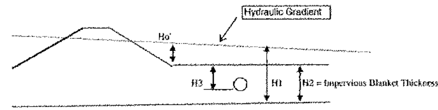
Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev.	777
Ground Elev.	760
Blanket Base Elev.	748
Bedrock Elev.	671



0	Berm Width, Wt (feet)
0	Berm Height, t (feet)
25	Pipe Dept (ft)
735	Pipe Elev.
48	Pipe Diameter (in)
115	Soil Unit Weight, γsoil (pcf)
25	Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

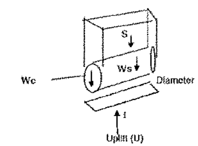
Dist from toe FT	Ho FT
0	7
500	4

No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 48 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	735	748	760	25	12	19.0	2470.0	191	784	9660	9880	0	1.08	1.00
25	6.9	735	748	760	25	12	18.9	2450.5	191	784	9660	9802	0	1.08	1.00
50	6.7	735	748	760	25	12	18.7	2431.0	191	784	9660	9724	0	1.09	1.01
100	6.4	735	748	760	25	12	18.4	2392.0	191	784	9660	9568	0	1.11	1.03
125	6.3	735	748	760	25	12	18.3	2372.5	191	784	9660	9490	0	1.12	1.04
150	6.1	735	748	760	25	12	18.1	2353.0	191	784	9660	9412	0	1.13	1.05
200	5.8	735	748	760	25	12	17.8	2314.0	191	784	9660	9256	0	1.15	1.06
300	5.2	735	748	760	25	12	17.2	2236.0	191	784	9660	8944	0	1.19	1.10
400	4.6	735	748	760	25	12	16.6	2158.0	191	784	9660	8632	0	1.23	1.14
500	4.0	735	748	760	25	12	16.0	2080.0	191	784	9660	8320	0	1.28	1.18

Ws Calc 48 -in steel 1lf
191
Wc Calc
784

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 190.70 lb per ft (48-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 780 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



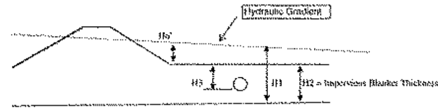
U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 48
There is uplift concern at a depth of 25 feet or less within 200 feet landward of the levee

Annandale Area
12
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev. [illegible]
Ground Elev. 760
Blanket Base Elev. 748
Bedrock Elev. [illegible]

0 Berm Width, Wt (feet)
0 Berm Height, t (feet)
35 Pipe Depth (ft)
725 Pipe Elev.
48 Pipe Diameter (in)
Ground Elev = Landside Average Ground Surface Used as Reference Datum
[illegible] Soil Unit Weight, γsoil (pcf)
35 Pipe Depth (Check)



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

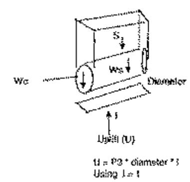
Dist from toe FT	Ho FT
0	7
500	4

No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 48 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SF (Full)	SF (Empty)
0	7.0	725	748	760	35	12	19.0	3458.0	191	784	14260	13832	0	[illegible]	1.04
25	6.9	725	748	760	35	12	18.9	3430.7	191	784	14260	13723	0	[illegible]	1.05
50	6.7	725	748	760	35	12	18.7	3403.4	191	784	14260	13614	0	[illegible]	1.06
100	6.4	725	748	760	35	12	18.4	3348.8	191	784	14260	13395	0	[illegible]	1.08
125	6.3	725	748	760	35	12	18.3	3321.5	191	784	14260	13286	0	[illegible]	1.09
150	6.1	725	748	760	35	12	18.1	3294.2	191	784	14260	13177	0	[illegible]	1.10
200	5.8	725	748	760	35	12	17.8	3239.6	191	784	14260	12958	0	[illegible]	[illegible]
300	5.2	725	748	760	35	12	17.2	3130.4	191	784	14260	12522	0	[illegible]	[illegible]
400	4.6	725	748	760	35	12	16.6	3021.2	191	784	14260	12085	0	[illegible]	[illegible]
500	4.0	725	748	760	35	12	16.0	2912.0	191	784	14260	11648	0	[illegible]	[illegible]

Ws Calc
48 -in steel lit
191
Wc Calc
784

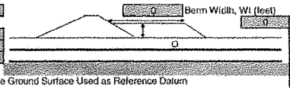
See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 190.70 lb per ft (48-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 780 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 48
There is uplift concern at a depth of 35 feet or less within 150 feet landward of the levee

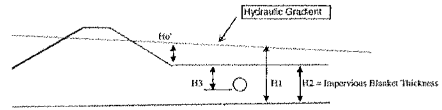
Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev.	777	
Ground Elev.	760	
Blanket Base Elev.	748	
Bedrock Elev.	671	



0	Berm Width, Wt (feet)
0	Berm Height, t (feet)
45	Pipe Depth (ft)
715	Pipe Elev.
48	Pipe Diameter (in)
116	Soil Unit Weight, γsoil (pcf)
45	Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

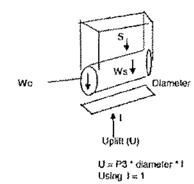
Dist from toe FT	Ho FT
0	7
500	4

No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 48 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	715	748	760	45	12	19.0	4446.0	191	784	18860	17784	0	1.12	1.07
25	6.9	715	748	760	45	12	18.9	4410.9	191	784	18860	17644	0	1.12	1.08
50	6.7	715	748	760	45	12	18.7	4375.8	191	784	18860	17503	0	1.13	1.09
100	6.4	715	748	760	45	12	18.4	4305.6	191	784	18860	17222	0	1.15	1.11
125	6.3	715	748	760	45	12	18.3	4270.5	191	784	18860	17082	0	1.16	1.12
150	6.1	715	748	760	45	12	18.1	4235.4	191	784	18860	16942	0	1.17	1.12
200	5.8	715	748	760	45	12	17.8	4165.2	191	784	18860	16661	0	1.19	1.14
300	5.2	715	748	760	45	12	17.2	4024.8	191	784	18860	16099	0	1.22	1.18
400	4.6	715	748	760	45	12	16.6	3884.4	191	784	18860	15538	0	1.26	1.23
500	4.0	715	748	760	45	12	16.0	3744.0	191	784	18860	14976	0	1.32	1.27

Ws Calc
48 -in steel 1ll
191
Wc Calc
784

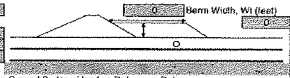
See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 190.70 lb per ft (48-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 780 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



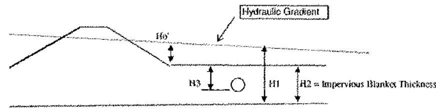
Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 48
There is uplift concern at a depth of 45 feet or less within 50 feet landward of the levee

Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev.	773	
Ground Elev.	760	
Blanket Base Elev.	748	
Bedrock Elev.	671	
Berm Width, Wt (feet)	0	
Berm Height, t (feet)	0	
Pipe Depth (ft)	55	
Pipe Elev.	705	
Pipe Diameter (in)	48	
Soil Unit Weight, γsoil (pcf)	115	
Pipe Depth (Check)	55	



Ground Elev = Landside Average Ground Surface Used as Reference Datum



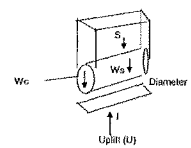
From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

Dist from toe FT	Ho FT
0	7
500	4

d=Landside Distance from Toe Feet	No Berm Calcs With WELLS Excess Head Feet, Ho'	Invert Elev of pipe 48-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	705	748	760	55	12	19.0	5434.0	191	784	23460	21736	0	1.12	1.09
25	6.9	705	748	760	55	12	18.9	5391.1	191	784	23460	21564	0	1.13	1.10
50	6.7	705	748	760	55	12	18.7	5348.2	191	784	23460	21393	0	1.14	1.11
100	6.4	705	748	760	55	12	18.4	5262.4	191	784	23460	21050	0	1.16	1.12
125	6.3	705	748	760	55	12	18.3	5219.5	191	784	23460	20878	0	1.17	1.13
150	6.1	705	748	760	55	12	18.1	5176.6	191	784	23460	20706	0	1.18	1.14
200	5.8	705	748	760	55	12	17.8	5090.8	191	784	23460	20363	0	1.20	1.16
300	5.2	705	748	760	55	12	17.2	4919.2	191	784	23460	19677	0	1.24	1.20
400	4.6	705	748	760	55	12	16.6	4747.6	191	784	23460	18990	0	1.29	1.25
500	4.0	705	748	760	55	12	16.0	4576.0	191	784	23460	18304	0	1.33	1.29

Ws Calc
48-in steel 1ft
191
Wc Calc
784

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 190.70 lb per ft (48-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 780 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

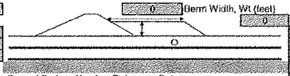


U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 48
There is uplift concern at a depth of 55 feet or less within 25 feet landward of the levee

Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev. 777



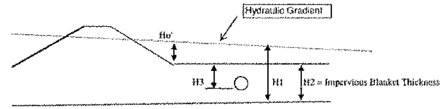
Berm Width, Wt (feet) 0
Berm Height, t (feet) 0

Ground Elev. 760
Blanket Base Elev. 748
Bedrock Elev. 671

15 Pipe Depth (ft)
745 Pipe Elev.
72 Pipe Diameter (in)

Ground Elev = Landside Average Ground Surface Used as Reference Datum

115 Soil Unit Weight, γsoil (pcf)
15 Pipe Depth (Check)



From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

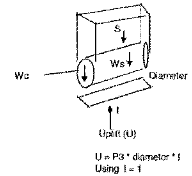
Dist from toe FT	Ho FT
0	7
500	4

No Berm Calcs With WELLS

d=Landside Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 72-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	745	748	760	15	12	19.0	1482.0	1532	1764	6210	8892	0	1.07	0.87
25	6.9	745	748	760	15	12	18.9	1470.3	1532	1764	6210	8822	0	1.08	0.88
50	6.7	745	748	760	15	12	18.7	1458.6	1532	1764	6210	8752	0	1.09	0.88
100	6.4	745	748	760	15	12	18.4	1435.2	1532	1764	6210	8611	0	1.10	0.90
125	6.3	745	748	760	15	12	18.3	1423.5	1532	1764	6210	8541	0	1.11	0.91
150	6.1	745	748	760	15	12	18.1	1411.8	1532	1764	6210	8471	0	1.12	0.91
200	5.8	745	748	760	15	12	17.8	1388.4	1532	1764	6210	8330	0	1.14	0.93
300	5.2	745	748	760	15	12	17.2	1341.6	1532	1764	6210	8050	0	1.18	0.96
400	4.6	745	748	760	15	12	16.6	1294.8	1532	1764	6210	7769	0	1.22	1.00
500	4.0	745	748	760	15	12	16.0	1248.0	1532	1764	6210	7488	0	1.27	1.03

Ws Calc
72-in RCP 1lf
1532
Wc Calc
1764

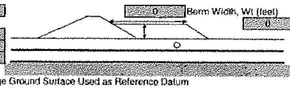
See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 1532 lb per ft (72-inch Diameter RCP, 6" wall thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 1764 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 72
There is uplift concern at a depth of 15 feet or less within 500 feet landward of the levee

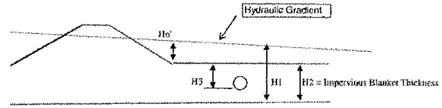
Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev. 777
Ground Elev. 760
Blanket Base Elev. 748
Bedrock Elev. 671



0 Berm Width, Wt (feet)
0 Berm Height, t (feet)
25 Pipe Depth (ft)
735 Pipe Elev.
72 Pipe Diameter (in)
115 Soil Unit Weight, γsoil (pcf)
25 Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum



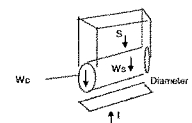
From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

Dist from toe FT	Ho FT
0	7
500	4

d=Landside	No Berm Calcs With WELLS														
Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 72 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	735	748	760	25	12	19.0	2470.0	1532	1764	13110	14820	0	1.11	0.99
25	6.9	735	748	760	25	12	18.9	2450.5	1532	1764	13110	14703	0	1.12	1.00
50	6.7	735	748	760	25	12	18.7	2431.0	1532	1764	13110	14586	0	1.12	1.00
100	6.4	735	748	760	25	12	18.4	2392.0	1532	1764	13110	14352	0	1.14	1.02
125	6.3	735	748	760	25	12	18.3	2372.5	1532	1764	13110	14235	0	1.15	1.03
150	6.1	735	748	760	25	12	18.1	2353.0	1532	1764	13110	14118	0	1.16	1.04
200	5.8	735	748	760	25	12	17.8	2314.0	1532	1764	13110	13884	0	1.18	1.05
300	5.2	735	748	760	25	12	17.2	2236.0	1532	1764	13110	13416	0	1.22	1.09
400	4.6	735	748	760	25	12	16.6	2158.0	1532	1764	13110	12948	0	1.27	1.13
500	4.0	735	748	760	25	12	16.0	2080.0	1532	1764	13110	12480	0	1.31	1.17

Ws Calc 72 -in RCP lf
1532
Wc Calc 1764

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 1532 lb per ft (72-inch Diameter RCP, 6" wall thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 1764 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



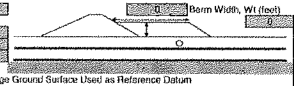
U = P3 * diameter * I
Using I = 1

Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 72
There is uplift concern at a depth of 25 feet or less within 300 feet landward of the levee

Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev. 777

Ground Elev. 760
Blanket Base Elev. 748
Bedrock Elev. 671



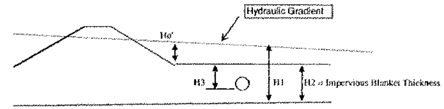
0 Berm Width, Wt (feet)
0 Berm Height, t (feet)
35 Pipe Depth (ft)
725 Pipe Elev.
72 Pipe Diameter (in)

115 Soil Unit Weight, γsoil (pcf)
35 Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum

From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

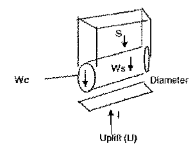
Dist from toe FT	Ho FT
0	7
500	4



d=Landside	No Berm Calcs With WELLS														
Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 72 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	725	748	760	35	12	19.0	3458.0	1532	1764	20010	20748	0	1.12	1.04
25	6.9	725	748	760	35	12	18.9	3430.7	1532	1764	20010	20584	0	1.13	1.05
50	6.7	725	748	760	35	12	18.7	3403.4	1532	1764	20010	20420	0	1.14	1.05
100	6.4	725	748	760	35	12	18.4	3348.8	1532	1764	20010	20093	0	1.16	1.07
125	6.3	725	748	760	35	12	18.3	3321.5	1532	1764	20010	19929	0	1.17	1.08
150	6.1	725	748	760	35	12	18.1	3294.2	1532	1764	20010	19765	0	1.18	1.09
200	5.8	725	748	760	35	12	17.8	3239.6	1532	1764	20010	19438	0	1.20	1.11
300	5.2	725	748	760	35	12	17.2	3130.4	1532	1764	20010	18782	0	1.24	1.15
400	4.6	725	748	760	35	12	16.6	3021.2	1532	1764	20010	18127	0	1.29	1.19
500	4.0	725	748	760	35	12	16.0	2912.0	1532	1764	20010	17472	0	1.33	1.23

Ws Calc
72 -in RCP tif
1532
Wc Calc
1764

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 1532 lb per ft (72-inch Diameter RCP, 6" wall thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 1764 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



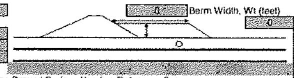
U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 72
There is uplift concern at a depth of 35 feet or less within 150 feet landward of the levee

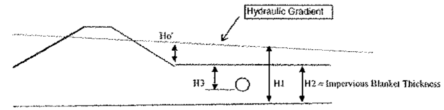
Armourdale Area
L2
Stationing Range
66+00
79+00
500-yr + 3ft

Levee Elev. 777
Ground Elev. 760
Blanket Base Elev. 748
Bedrock Elev. 671

0 Berm Width, Wt (feet)
0 Berm Height, t (feet)
45 Pipe Depth (ft)
715 Pipe Elev.
72 Pipe Diameter (in)
115 Soil Unit Weight, γsoil (pcf)
45 Pipe Depth (Check)



Ground Elev = Landside Average Ground Surface Used as Reference Datum



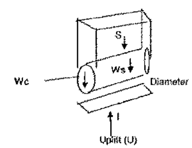
From Geotechnical Seepage Calculations with Cutoff Wall (2-27-2007)

Dist from toe FT	Ho FT
0	7
500	4

d=Landside Distance from Toe Feet	No Berm Calcs With WELLS Excess Head Feet, Ho'	Invert Elev of pipe 72 -in RCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.0	715	748	760	45	12	19.0	4446.0	1532	1764	26910	26676	0	1.13	1.07
25	6.9	715	748	760	45	12	18.9	4410.9	1532	1764	26910	26465	0	1.14	1.07
50	6.7	715	748	760	45	12	18.7	4375.8	1532	1764	26910	26255	0	1.15	1.08
100	6.4	715	748	760	45	12	18.4	4305.6	1532	1764	26910	25834	0	1.17	1.10
125	6.3	715	748	760	45	12	18.3	4270.5	1532	1764	26910	25623	0	1.18	1.11
150	6.1	715	748	760	45	12	18.1	4235.4	1532	1764	26910	25412	0	1.19	1.12
200	5.8	715	748	760	45	12	17.8	4165.2	1532	1764	26910	24991	0	1.21	1.14
300	5.2	715	748	760	45	12	17.2	4024.8	1532	1764	26910	24149	0	1.25	1.18
400	4.6	715	748	760	45	12	16.6	3884.4	1532	1764	26910	23306	0	1.30	1.22
500	4.0	715	748	760	45	12	16.0	3744.0	1532	1764	26910	22464	0	1.34	1.27

Ws Calc
72 -in RCP 1lf
1532
Wc Calc
1764

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 1532 lb per ft (72-inch Diameter RCP, 6" wall thick) (Am. Conc. Pipe Ass.)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((24/12)^2) *1*62.4 = 1764 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 66+00 to 79+00
For pipe diam (in): 72
There is uplift concern at a depth of 45 feet or less within 50 feet landward of the levee

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L3																							feet	
Stationing Range 79+00	350	350	30	30	30	63	15	0.84	65	0	2.03	65	100	813	978	978	12.47	12.47	12.47	0.42	813	813	0	0.000
190+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 65 — Levee Width, L2 100

Levee Elev. 775 — Berm Width, Wt (feet) 0

Berm Height, t (feet) 0

Ground Elev. 760 — Pipe Depth (ft) 3.33

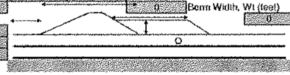
Blanket Base Elev. 730 — Pipe Elev. 756.67

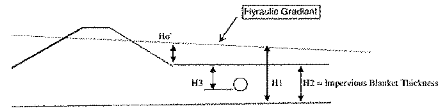
Bedrock Elev. 667 — Pipe Diameter (in) 6

Soil Type

Soil Unit Weight, γsoil (pcf) 115

3.33 Pipe Depth (Check)





d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 6-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	12.47	756.67	730	760	3.33	30	42.5	294.2	19	12	163	147	0	1.32	1.24
25	12.09	756.67	730	760	3.33	30	42.1	291.6	19	12	163	146	0	1.33	1.25
50	11.73	756.67	730	760	3.33	30	41.7	289.0	19	12	163	145	0	1.34	1.26
100	11.03	756.67	730	760	3.33	30	41.0	284.2	19	12	163	142	0	1.36	1.28
125	10.70	756.67	730	760	3.33	30	40.7	281.9	19	12	163	141	0	1.38	1.29
150	10.37	756.67	730	760	3.33	30	40.4	279.6	19	12	163	140	0	1.39	1.30
200	9.75	756.67	730	760	3.33	30	39.8	275.3	19	12	163	138	0	1.41	1.32
300	8.62	756.67	730	760	3.33	30	38.6	267.5	19	12	163	134	0	1.45	1.36
400	7.63	756.67	730	760	3.33	30	37.6	260.6	19	12	163	130	0	1.49	1.39
500	6.74	756.67	730	760	3.33	30	36.7	254.5	19	12	163	127	0	1.52	1.43

Ws Calc 6-in steel 1lf
19
Wc Calc 12

See Sample Calculations for list of abbreviations and sample calculations

Ws = weight of structure per foot of length = 18.97 lb per ft (6-inch Diameter Steel Pipe, .280" thick)

Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((3/12)^2) *1*62.4 = 12 plf

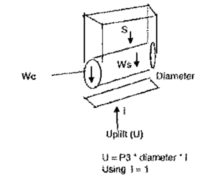
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil

P3 = H3*(H1/H2)*γwater

U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3

Wg = weight of surcharge water above top surface of structure control by gravity flow

SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Use this table to determine the distance from toe at which the Safety Factor is met

Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 760	Distance from toe (ft) 0				Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 19		Wc = 12 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	760	0		19	12		
3.33	756.67	163	147	19	12	1.32	1.24
4.33	755.67	220	191	19	12	1.31	1.25
5.33	754.67	278	235	19	12	1.31	1.26
6.33	753.67	335	280	19	12	1.31	1.27
7.33	752.67	393	324	19	12	1.31	1.27
8.33	751.67	450	368	19	12	1.31	1.28
9.33	750.67	508	412	19	12	1.31	1.28
10.33	749.67	565	456	19	12	1.31	1.28
11.33	748.67	623	500	19	12	1.31	1.28

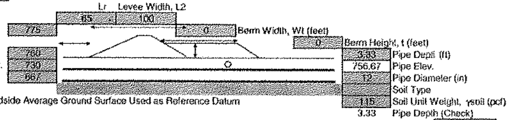
Conclusions

For levee stretch: 79+00 to 190+00

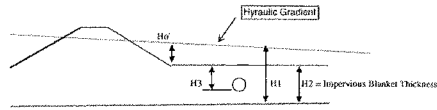
For pipe diam (in): 6

There is no uplift concern at a depth of 3.33 feet or deeper within 500 feet landward of the levee

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Lc	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L3																							feet	
Stationing Range																								
79+00	350	350	30	30	30	63	15	0.84	65	0	2.03	65	100	813	978	978	12.47	12.47	12.47	0.42	813	813	0	0.000
190+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								



Ground Elev = Landside Average Ground Surface Used as Reference Datum



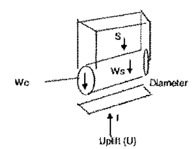
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	12.47	756.67	730	760	3.33	30	42.5	294.2	50	49	268	294	0	1.25	1.08
25	12.09	756.67	730	760	3.33	30	42.1	291.6	50	49	268	292	0	1.26	1.09
50	11.73	756.67	730	760	3.33	30	41.7	289.0	50	49	268	289	0	1.27	1.10
100	11.03	756.67	730	760	3.33	30	41.0	284.2	50	49	268	284	0	1.29	1.12
125	10.70	756.67	730	760	3.33	30	40.7	281.9	50	49	268	282	0	1.30	1.13
150	10.37	756.67	730	760	3.33	30	40.4	279.6	50	49	268	280	0	1.31	1.14
200	9.75	756.67	730	760	3.33	30	39.8	275.3	50	49	268	275	0	1.33	1.15
300	8.62	756.67	730	760	3.33	30	38.6	267.5	50	49	268	268	0	1.37	1.19
400	7.63	756.67	730	760	3.33	30	37.6	260.6	50	49	268	261	0	1.41	1.22
500	6.74	756.67	730	760	3.33	30	36.7	254.5	50	49	268	255	0	1.44	1.25

Ws Calc
12 -in steel 1lf
50
Wc Calc
49

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 760	Ws = 50	Distance from toe (ft) 0	Wc = 49 Full		Uplift Safety Extreme Case (1.1 REQ'D)	
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	760	0		50	49		
3.33	756.67	268	294	50	49	1.25	1.08
4.33	755.67	383	383	50	49	1.26	1.13
5.33	754.67	498	471	50	49	1.27	1.16
6.33	753.67	613	559	50	49	1.27	1.18
7.33	752.67	728	648	50	49	1.28	1.20
8.33	751.67	843	736	50	49	1.28	1.21
9.33	750.67	958	824	50	49	1.28	1.22
10.33	749.67	1073	913	50	49	1.28	1.23
11.33	748.67	1188	1001	50	49	1.29	1.24

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 50 lb per ft (12-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((6/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 79+00 to 190+00
For pipe diam (in): 12
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 50 feet landward of the levee
There is uplift concern at a depth of 3.33 feet and less at the landward toe

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Page 1 of 2

MARCH 8 ,2007

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L3																							feet	
Stationing Range																								
79+00	350	350	30	30	30	63	15	0.84	65	0	2.03	65	100	813	978	978	12.47	12.47	12.47	0.42	813	813	0	0.000
190+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 65 | Levee Width, L2 100

Levee Elev. 775

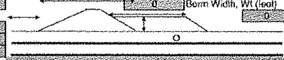
Berm Width, Wt (feet) 0

Berm Height, t (feet) 0

Ground Elev. 760

Blanket Base Elev. 730

Bedrock Elev. 667



10 Pipe Depth (ft)

750 Pipe Elev.

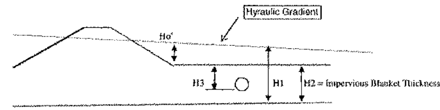
12 Pipe Diameter (in)

Soil Type

115 Soil Unit Weight, γsoil (pcf)

10 Pipe Depth (Check)

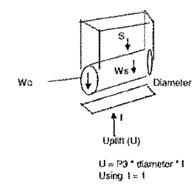
Ground Elev = Landside Average Ground Surface Used as Reference Datum



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	12.47	750	730	760	10	30	42.5	883.4	50	49	1035	883	0	1.28	1.23
25	12.09	750	730	760	10	30	42.1	875.6	50	49	1035	876	0	1.29	1.24
50	11.73	750	730	760	10	30	41.7	867.9	50	49	1035	868	0	1.31	1.25
100	11.03	750	730	760	10	30	41.0	853.4	50	49	1035	853	0	1.33	1.27
125	10.70	750	730	760	10	30	40.7	846.5	50	49	1035	846	0	1.34	1.28
150	10.37	750	730	760	10	30	40.4	839.7	50	49	1035	840	0	1.35	1.29
200	9.75	750	730	760	10	30	39.8	826.9	50	49	1035	827	0	1.37	1.31
300	8.62	750	730	760	10	30	38.6	803.4	50	49	1035	803	0	1.41	1.35
400	7.63	750	730	760	10	30	37.6	782.6	50	49	1035	783	0	1.45	1.39
500	6.74	750	730	760	10	30	36.7	764.3	50	49	1035	764	0	1.48	1.42

Ws Calc
12 -in steel 1lf
50
Wc Calc
49

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 50 lb per ft (12-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((6/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 760		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 50		Wc = 49 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	760	0		50	49		
10	750	1035	883	50	49	1.28	1.23
11	749	1150	972	50	49	1.28	1.23
12	748	1265	1060	50	49	1.29	1.24
13	747	1380	1148	50	49	1.29	1.24
14	746	1495	1237	50	49	1.29	1.25
15	745	1610	1325	50	49	1.29	1.25
16	744	1725	1413	50	49	1.29	1.26
17	743	1840	1502	50	49	1.29	1.26
18	742	1955	1590	50	49	1.29	1.26

Conclusions
For levee stretch: 79+00 to 190+00
For pipe diam (in): 12
There is no uplift concern at a depth of 10 feet or deeper within 500 feet landward of the levee

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L3																							foot	
Stationing Range																								
79+00	350	350	30	30	30	63	15	0.84	65	0	2.03	65	100	813	978	978	12.47	12.47	12.47	0.42	813	813	0	0.000
190+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 65 Levee Width, L2 100

Levee Elev. 775

0 Berm Width, Wt (feet)

0 Berm Height, t (feet)

Ground Elev. 760

Blanket Base Elev. 730

Bedrock Elev. 667

3.33 Pipe Depth (ft)

756.67 Pipe Elev.

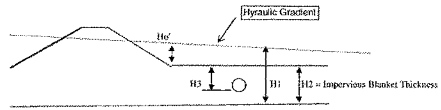
16 Pipe Diameter (in)

Soil Type

115 Soil Unit Weight, γsoil (pcf)

3.33 Pipe Depth (Check)

Ground Elev = Landside Average Ground Surface Used as Reference Datum



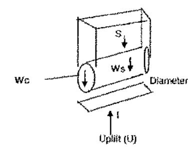
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 16-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	12.47	756.67	730	760	3.33	30	42.5	294.2	63	87	306	392	0	1.16	0.94
25	12.09	756.67	730	760	3.33	30	42.1	291.6	63	87	306	389	0	1.17	0.95
50	11.73	756.67	730	760	3.33	30	41.7	289.0	63	87	306	385	0	1.18	0.96
100	11.03	756.67	730	760	3.33	30	41.0	284.2	63	87	306	379	0	1.20	0.97
125	10.70	756.67	730	760	3.33	30	40.7	281.9	63	87	306	376	0	1.21	0.98
150	10.37	756.67	730	760	3.33	30	40.4	279.6	63	87	306	373	0	1.22	0.99
200	9.75	756.67	730	760	3.33	30	39.8	275.3	63	87	306	367	0	1.24	1.01
300	8.62	756.67	730	760	3.33	30	38.6	267.5	63	87	306	357	0	1.28	1.03
400	7.63	756.67	730	760	3.33	30	37.6	260.6	63	87	306	347	0	1.31	1.06
500	6.74	756.67	730	760	3.33	30	36.7	254.5	63	87	306	339	0	1.34	1.09

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 760	Distance from toe (ft) 0				Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 63		Wc = 87 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	760	0		63	87		
3.33	756.67	306	392	63	87	1.16	0.94
4.33	755.67	459	510	63	87	1.20	1.02
5.33	754.67	613	628	63	87	1.22	1.08
6.33	753.67	766	746	63	87	1.23	1.11
7.33	752.67	919	863	63	87	1.24	1.14
8.33	751.67	1073	981	63	87	1.25	1.16
9.33	750.67	1226	1099	63	87	1.25	1.17
10.33	749.67	1379	1217	63	87	1.26	1.19
11.33	748.67	1533	1335	63	87	1.26	1.20

Ws Calc 16-in steel 1ft
63
Wc Calc
87

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 62.58 lb per ft (16-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((8/12)^2) *1*62.4 = 49 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * t
Using t = 1

Conclusions
For levee stretch: 79+00 to 190+00
For pipe diam (in): 16
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 5.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DhL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L3																							feet	
Stationing Range																								
79+00	350	350	30	30	30	63	15	0.84	65	0	2.03	65	100	813	978	978	12.47	12.47	12.47	0.42	813	813	0	0.000
190+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 65 | Levee Width, L2 100

Levee Elev. 775 | Berm Width, Wt (feet) 0 | Berm Height, t (feet) 0

Ground Elev. 760 | 10 Pipe Depth (ft)

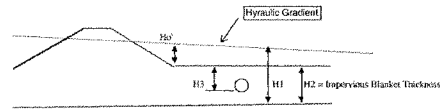
Blanket Base Elev. 730 | 750 Pipe Elev.

Bedrock Elev. 667 | 16 Pipe Diameter (in)

Soil Type

Ground Elev = Landside Average Ground Surface Used as Reference Datum | 115 Soil Unit Weight, γsoil (pcf)

10 Pipe Depth (Check)



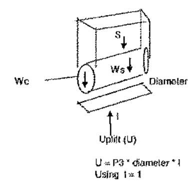
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 16 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	12.47	750	730	760	10	30	42.5	883.4	63	87	1329	1178	0	1.26	1.18
25	12.09	750	730	760	10	30	42.1	875.6	63	87	1329	1167	0	1.27	1.19
50	11.73	750	730	760	10	30	41.7	867.9	63	87	1329	1157	0	1.28	1.20
100	11.03	750	730	760	10	30	41.0	853.4	63	87	1329	1138	0	1.30	1.22
125	10.70	750	730	760	10	30	40.7	846.5	63	87	1329	1129	0	1.31	1.23
150	10.37	750	730	760	10	30	40.4	839.7	63	87	1329	1120	0	1.32	1.24
200	9.75	750	730	760	10	30	39.8	826.9	63	87	1329	1102	0	1.34	1.26
300	8.62	750	730	760	10	30	38.6	803.4	63	87	1329	1071	0	1.38	1.30
400	7.63	750	730	760	10	30	37.6	782.6	63	87	1329	1044	0	1.42	1.33
500	6.74	750	730	760	10	30	36.7	764.3	63	87	1329	1019	0	1.45	1.37

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 760		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 63		Wc = 87 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	760	0		63	87		
10	750	1329	1178	63	87	1.26	1.18
11	749	1482	1296	63	87	1.26	1.19
12	748	1636	1413	63	87	1.26	1.20
13	747	1789	1531	63	87	1.27	1.21
14	746	1942	1649	63	87	1.27	1.22
15	745	2096	1767	63	87	1.27	1.22
16	744	2249	1885	63	87	1.27	1.23
17	743	2402	2002	63	87	1.27	1.23
18	742	2556	2120	63	87	1.26	1.24

Ws Calc 16 -in steel 1ft
63
Wc Calc 87

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 62.58 lb per ft (16-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((8/12)^2) *1*62.4 = 49 pif
S = surcharge loads = weight of saturated soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



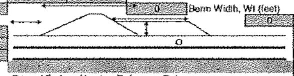
Conclusions
For levee stretch: 79+00 to 190+00
For pipe diam (in): 16
There is no uplift concern at a depth of 3.33 feet or deeper within 500 feet landward of the levee

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L3																							feet	
Stationing Range																								
79+00	350	350	30	30	30	63	15	0.84	65	0	2.03	65	100	813	978	978	12.47	12.47	12.47	0.42	813	813	0	0.000
190+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

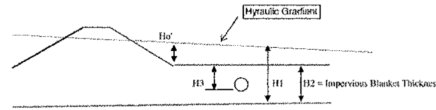
Lr 65 — Levee Width, L2 100

Levee Elev. 775 — Berm Width, Wt (feet) 0

Berm Height, t (feet) 0



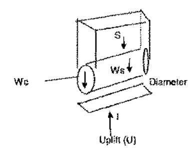
Ground Elev.	760	3.33	Pipe Depth (ft)
Blanket Base Elev.	730	756.67	Pipe Elev.
Bedrock Elev.	667	24	Pipe Diameter (in)
			Soil Type
Ground Elev = Landside Average Ground Surface Used as Reference Datum		115	Soil Unit Weight, γsoil (pcf)
		3.33	Pipe Depth (Check)



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 24 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	12.47	756.67	730	760	3.33	30	42.5	294.2	95	196	306	588	0	1.01	0.68
25	12.09	756.67	730	760	3.33	30	42.1	291.6	95	196	306	583	0	1.02	0.69
50	11.73	756.67	730	760	3.33	30	41.7	289.0	95	196	306	578	0	1.03	0.69
100	11.03	756.67	730	760	3.33	30	41.0	284.2	95	196	306	568	0	1.05	0.71
125	10.70	756.67	730	760	3.33	30	40.7	281.9	95	196	306	564	0	1.06	0.71
150	10.37	756.67	730	760	3.33	30	40.4	279.6	95	196	306	559	0	1.07	0.72
200	9.75	756.67	730	760	3.33	30	39.8	275.3	95	196	306	551	0	1.08	0.73
300	8.62	756.67	730	760	3.33	30	38.6	267.5	95	196	306	535	0	1.12	0.75
400	7.63	756.67	730	760	3.33	30	37.6	260.6	95	196	306	521	0	1.15	0.77
500	6.74	756.67	730	760	3.33	30	36.7	254.5	95	196	306	509	0	1.17	0.79

Ws Calc
24 -in steel 1lf
95
Wc Calc
196

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 94.62 lb per ft (24-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((12/12)^2) *1*62.4 = 196 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 760		Distance from toe (ft) 0				
		Ws = 95		Wc = 196 Full		Uplift Safety Extreme Case (1.1 REQ'D)	
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	760	0		95	196		
3.33	756.67	306	588	95	196	1.01	0.68
4.33	755.67	536	765	95	196	1.08	0.82
5.33	754.67	766	942	95	196	1.12	0.91
6.33	753.67	996	1118	95	196	1.15	0.98
7.33	752.67	1226	1295	95	196	1.17	1.02
8.33	751.67	1456	1472	95	196	1.19	1.05
9.33	750.67	1686	1648	95	196	1.20	1.08
10.33	749.67	1916	1825	95	196	1.21	1.10
11.33	748.67	2146	2002	95	196	1.22	1.12

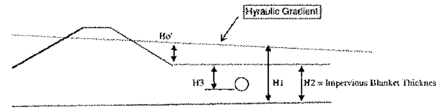
Conclusions
For levee stretch: 79+00 to 190+00
For pipe diam (in): 24
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 9.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	La	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L3																							feet	
Stationing Range																								
79+00	350	350	30	30	30	63	15	0.84	65	0	2.03	65	100	813	978	978	12.47	12.47	12.47	0.42	813	813	0	0.000
190+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr Levee Width, L2
65 100

Levee Elev. 775 — 0 Berm Width, Wt (feet)
0 Berm Height, t (feet)
Ground Elev. 760 — 10 Pipe Depth (ft)
Blanket Base Elev. 730 — 750 Pipe Elev.
Bedrock Elev. 667 — 24 Pipe Diameter (in)
Soil Type
115 Soil Unit Weight, γsoil (pcf)
10 Pipe Depth (Check)

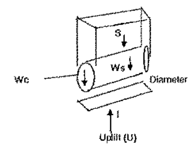
Ground Elev = Landside Average Ground Surface Used as Reference Datum



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 24-in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	12.47	750	730	760	10	30	42.5	883.4	95	196	1840	1767	0	1.21	1.10
25	12.09	750	730	760	10	30	42.1	875.6	95	196	1840	1751	0	1.22	1.11
50	11.73	750	730	760	10	30	41.7	867.9	95	196	1840	1736	0	1.23	1.11
100	11.03	750	730	760	10	30	41.0	853.4	95	196	1840	1707	0	1.25	1.13
125	10.70	750	730	760	10	30	40.7	846.5	95	196	1840	1693	0	1.26	1.14
150	10.37	750	730	760	10	30	40.4	839.7	95	196	1840	1679	0	1.27	1.15
200	9.75	750	730	760	10	30	39.8	826.9	95	196	1840	1654	0	1.29	1.17
300	8.62	750	730	760	10	30	38.6	803.4	95	196	1840	1607	0	1.33	1.20
400	7.63	750	730	760	10	30	37.6	782.6	95	196	1840	1565	0	1.36	1.24
500	6.74	750	730	760	10	30	36.7	764.3	95	196	1840	1529	0	1.39	1.27

Ws Calc
24-in steel 1ft
95
Wc Calc
196

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 94.62 lb per ft (24-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((12/12)^2) *1*62.4 = 196 plf
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



Uplift (U)

U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is met
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 760		Distance from toe (ft) 0				
		Ws = 95		Wc = 196 Full		Uplift Safety Extreme Case (1.1 REQ'D)	
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	760	0		95	196		
10	750	1840	1767	95	196	1.21	1.10
11	749	2070	1944	95	196	1.21	1.11
12	748	2300	2120	95	196	1.22	1.13
13	747	2530	2297	95	196	1.23	1.14
14	746	2760	2474	95	196	1.23	1.15
15	745	2990	2650	95	196	1.24	1.16
16	744	3220	2827	95	196	1.24	1.17
17	743	3450	3004	95	196	1.25	1.18
18	742	3680	3180	95	196	1.25	1.19

Conclusions
For levee stretch: 79+00 to 190+00
For pipe diam (in): 24
There is uplift concern at a depth of 40 inches (3.3 feet) and less at the toe of the levee
There is uplift concern at a depth of 10 feet and less at the landward toe

Kansas Citys, Missouri and Kansas
Flood Risk Management Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Final Feasibility Report

Chapter A-7

CIVIL DESIGN
CID

CHAPTER A-7
CIVIL DESIGN – CID

A-7.1 SITE SELECTION AND PROJECT DEVELOPMENT

A-7.1.1 Introduction

This chapter of the engineering appendix presents the results of the civil design evaluation performed as part of the future conditions analysis for the Central Industrial District (CID) Unit of the Kansas Citys, Missouri and Kansas, Flood Risk Management Project. The U.S. Army Corps of Engineers (USACE), Kansas City District designed and constructed the Kansas Citys protection system. This portion of the study considers raises on the CID Unit to the 0.2% (500-year)-plus-3-ft, water surface profile elevation, hereafter referred to as N500+3.

The CID-Kansas Unit is located in Wyandotte County, Kansas, along the right bank of the Kansas River RM 3.1 (Approximately Turkey Creek outfall) to RM 367.3 of the Missouri River, near the Wastewater Treatment Facility of the Unified Government. The flood protection unit includes levees, floodwalls, riprap, and levee toe protection, toe drains along the concrete floodwalls, surfaced levee crown, ramps, and turnouts, seeded landside slopes of levees, stop log gaps, sandbag gaps, drainage structures, relief wells, and pumping plants. Stationing of the unit begins at the lower end (LE). The LE is one continuous levee with CID-Missouri. CID-Missouri ends with Station 89+37.34 which is equivalent to CID-Kansas Station 0+00. CID-Kansas Unit ends at Station 167+95.

A-7.1.2 Levee Footprint

The unit includes sections of levee and floodwall. All alternatives include widening the levee footprint, replacing floodwalls or placing a floodwall on top of existing levee. Stability berms, area fill, or underseepage features are needed as indicated by the geotechnical analysis. Refer to Exhibit A-7.1 for quantity calculations at the end of this chapter.

A-7.1.3 Borrow Area

Prospective borrow areas were identified by the Sponsor and screened through joint Corps and Sponsor efforts during the Phase 1 portion of this feasibility project. Regarding borrow area, Phase 1 focused on the Argentine Unit. The borrow area for CID Unit will be the same as for Argentine and Armourdale Units. Refer to Exhibit A-7.2, "Borrow Area for Proposed Argentine, Armourdale, and CID-KS Unit Raise" at the end of this chapter for a discussion of the borrow area for the project.

A-7.1.4 Haul Routes

Haul routes from the borrow site at WaterOne to various points along the levee/floodwall alignment were generated based on the limited access points to the levee/floodwall system along the CID-KS Unit. Exhibit A-7.3 in the Supplemental Exhibits section at the end of this chapter shows the haul routes and their distances from WaterOne to various access points along the CID-KS Unit.

A-7.2 REAL ESTATE CONSIDERATIONS

Research is being conducted by CENWK Real Estate Staff to complete Preliminary Attorney's Opinion of Compensability as utilities are identified. Any conclusion or categorization that an item is a utility or facility relocation would result in work to be performed at the cost of the non-federal sponsor as part of LERRD responsibilities and is preliminary only. During PED, the Government will make a final determination of the relocations necessary for the construction, operation or maintenances of the project after further analysis and completion and approval of Final Attorney's Opinions of Compensability for each of the impacted utilities and facilities. Further detail on all real estate issues, is discussed in the Real Estate Plan, Appendix C of the Feasibility Report. Also refer to the real estate plan for a discussion of real estate issues pertaining to borrow area.

A-7.3 UTILITY RELOCATIONS

A review of the Kansas City District's criteria for utility lines was performed. Based on discussions, a criteria document specific to this project was developed, and is shown as Exhibit A-7.4, "Kansas City's Levee and Floodwall Gravity and Utility Pipeline Guidance" at the end of this chapter. This document was used in determining the disposition of existing utility lines crossing the Central Industrial District, Kansas. Supporting calculations for utilities that require relocation can be found in Exhibit A-7.5 at the end of this chapter

A-7.3.1 Utility Levee Crossings

The study of utilities crossing the Central Industrial District, Kansas Unit was conducted to estimate costs for relocation or removal of functioning or abandoned utilities. Using the criteria indicated above, it was determined that most pressure pipelines currently passing under the levee would be relocated over the levee. Refer to Exhibit A-7.6, titled "CID-KS Utility and Drainage Crossings: Inventory and Action for N500+3 Raise", for the recommended action for each utility crossing the levee at the end of this chapter. Utilities will be relocated up and over the levee. The drawing in Exhibit A-7.7, titled "Typical Utility Crossing Levee Raise", displays typical utility crossings and is located in the supplemental exhibit section.

The utilities that require relocation are listed in Table A-7.1: Utilities that Require Relocation. Most of the utilities are currently located on an existing bridge structure that either penetrates through the top of the levee or near the base of the floodwall. It is anticipated that these utilities with either have to be extended beyond their existing penetration point for purposes of complying with the current guidance or for purposes of constructing the raise. The power lines at Station 124+60 service the Turkey Creek Sewer Gates. Currently this line penetrates under the floodwall. This line will be relocated up and over the floodwall.

Utility crossing recommendations are based on the N500+3 raise, therefore the quantities for the cost estimate is also based on the N500+3 raise. For the N500+0 and N500+5 alternatives, recommendations for utility crossings are the same as for N500+3. Cost estimates and quantities used for determining the N500+3 cost will be used for the N500+0 and the N500+5. There is negligible difference between the three raises in regards to recommended utility crossings. This approach for estimating the N500+0 and the N500+5 raise is similar to the approach taken in Phase 1 when cost estimating the Argentine Unit utility crossings.

TABLE A-7.1
Utilities that Require Relocation

Station (ft.)	Conduit Function
18+10	Fiber Optic
Approx 18+20	Fiber Optic
18+75	Fiber Optic
18+75	Fiber Optic
19+85	Fiber Optic and Railroad Communication
25+90	Gas
Approx 25+90	Oxygen
Approx 25+90	Gas
26+10	Gas
26+10	Water
57+10	Water
57+10	Power
57+50	Water
75+50	Fiber Optic
75+60	Fiber Optic
85+20	Water
104+50	Fiber Optic
Approx 124+60	Power

A-7.3.2 Special Design and Construction Considerations

The project team will conduct specific utilities relocation coordination and design planning prior to levee raise construction contract award. In recent projects, this relocation work has proven very problematic if not thoroughly scheduled and coordinated. Sponsors, and utility owners, are responsible for most utility relocations (for those utilities deemed without legal compensability), the Kansas City District must be consulted for approval of the relocation design and schedule. Detailed planning for utility relocations and assignment of responsibilities is fully developed by the latter stages of the PED phase. All parties (sponsor, utility owner, and Corps of Engineers) must prepare for a highly coordinated utility relocation effort as the levee raise begins.

Where lines are shown as abandoned or to be abandoned, the Kansas City's Levee and Floodwall Utility Crossings Criteria document will be followed.

A-7.3.2.1 Storm Drain, Utility, and Other Modifications due to Area Fill

Areas indicated as "area fill" located approximately 32+50 to 38+00; 63+00 to 74+75; and 77+00 and 94+50 will require the placement of soil and essentially raising the existing ground surface elevation. This will impact the drainage system, overhead power and lighting system, and a privately own billboard in these areas and modifications will have to be made. These areas are existing parking areas for adjacent businesses and temporary accommodations will have to be made. Power for the overhead lighting will be temporary relocated and put back when construction is finished. The high mast light and billboard will also have to be temporarily relocated. Supporting calculations are located in Exhibit A-7.8 at the end of this chapter for reference.

A-7.3.3 Power Lines

Large capacity power lines cross the CID-KS Unit. Of these large capacity lines, all cross the levee and the Kansas River and therefore have substantial structures holding them in place. These structures do not interfere with the proposed levee raise or the floodwall raise. Table A-7.2: Power Lines, is a list of the stations currently crossed by large capacity power lines and other lines running parallel to the levee.

TABLE A-7.2
Power Lines

Station (ft.)	Function
22+50	Large Capacity Power Lines
53+00	Power pole and guy wire (relocation)
53+50	Guy Wire (relocation)
60+00	Power & telephone (remove)
131+30	Large Capacity Power Lines
57+25 to approx 73+80	Power Lines (relocate)
Gateway 2000 Pump Plant to Field Pump House	Power Lines (remove)
Approx 120+00 to Approx 133+00	Power Lines (protect)

The existing clearance between most of the power lines and top of levee is approximately 40-ft although this will need to be verified during a land survey during detailed design phases. The N500+3 alternative results in a levee raise in the range of 3-ft to 5-ft. Coordination with the Board of Public Utilities (BPU) determined that the required clearance between the power lines and the levee is 20.9-ft. This clearance is based on the National Electric Safety Code (NESC). With the maximum raise of 5-ft reducing the minimum clearance to 35-ft, the clearance between the power lines and the levee is adequate.

Power lines running parallel (landward) with the levee will be protected, relocated, or removed during construction. Table A-7.2 summarizes these features. Supporting calculations are located in Exhibit A-7.9 at the end of this chapter for reference.

A-7.3.4 Utility Uplift

The study of uplift on existing utilities was conducted to estimate costs for relocation or removal of functioning or abandoned utilities. Utilities were identified in the critical zone for uplift concern, based on geotechnical and N500+3 conditions.

Water and sanitary sewer utility mapping was obtained from the Unified Government of Wyandotte County and the Board of Public Utilities. Storm sewers that do not cross the levee were not mapped because uplift is generally not a concern as long as both ends of the pipe are open to atmosphere. Being open to atmosphere at both ends allows the pipe to fill during inundation. Storm sewers are typically metal or concrete and are heavy enough not to float when filled with water. For the purposes of uplift it was assumed that underground electrical lines (UGE) were not affected. The information on natural gas lines and petroleum lines within the 500-ft zone was limited but was evaluated where the information was available. No known petroleum lines are located within the critical zone. Natural gas and water service lines to buildings are generally less than 6 inches in diameter and not located near the levee. Based upon previous uplift calculations, lines 6 inches and smaller are generally not affected by uplift, therefore only lines 6 inches and greater have been evaluated.

For this study, the CID-KS Unit was broken into ten segments for analysis with each segment having its own geotechnical features. The geotechnical input consisted of impervious blanket thickness and foundation sands thickness as well as levee dimensions, berm dimensions, bedrock depth, and soil density. Exhibit A-7.10 "CID-KS Utility Uplift Spreadsheet: Data Entry Worksheet" displays the existing geotechnical and dimensional conditions for each levee segment at the end of this chapter. For this study, the driving head of water represents the N500+3 level of protection. The locations of underseepage control features were considered in regards to uplift on utility lines. A factor of safety of 1.1 was used assuming each pipe is empty to determine uplift.

Exhibit A-7.11 contains a sample calculation for utility uplift at the end of this chapter. It shows how each of the geotechnical, dimensional, and hydraulic gradient inputs are used to calculate potential uplift concerns. The utility uplift spreadsheets for levees are based on this sample calculation. A set of spreadsheets was developed for each utility crossing the levee. Uplift was evaluated for each utility. If uplift was not a concern then no further evaluation was done on that pipe.

If uplift is found to be a concern, then uplift was evaluated further from the toe up to 500-ft. The distance from the levee at which the factor of safety equals 1.1 at the depth of the utility is the length of the utility that needs a remedy.

The results are summarized in Exhibit A-7.12, "Utility Uplift Summary", at the end of this chapter. The tables provide a list of utilities, indicating the levee segment, the size and type of line, and the length of line to be lowered or covered to alleviate uplift concern. The remedy developed to address uplift is concrete anchors.

Utility uplift recommendations are based on the N500+3 raise, therefore the quantities for the cost estimate is also based on the N500+3 raise. For the N500+0 and N500+5 alternatives, quantities for utility uplift are not the same as for N500+3. Quantities used for determining the N500+3 cost will be adjusted up for the N500+5 alternative and down for the N500+0 alternative. This approach for estimating the N500+0 and the N500+5 raise is similar to the approach taken in Phase 1 for estimating the Argentine Unit utility uplift recommendations. Based on Argentine quantities for uplift, the quantity of piping for the N500+0 raise will be calculated at 80% of the N500+3 quantity. The quantity of piping for the N500+5 raise will be calculated at 120% of the N500+3 quantity. The number of manholes to be replaced will remain the same for all alternatives, N500+0, N500+3, and N500+5.

A-7.3.5 Inspection Trench

Since the proposed construction primarily involves raising the existing protection, no specific locations for new inspection trenches are indicated or considered. In the event that conditions are encountered in the field which warrants investigation, an inspection trench may be used.

A-7.4 REFERENCES

1. American Water Works Association "Steel Pipe – A Guide for Design and Installation", AWWA M11 4, 2004.
2. Hydraulic Institute "Hydraulic Institute Engineering Data Book" Hydraulic Institute, Cleveland, Ohio.
3. EM 1110-2-1913, "Engineering and Design – Design and Construction of Levees"
4. Kansas City District regional specific guidance http://www.nwk.usace.army.mil/Missions/EngineeringDivision/GeotechnicalBranch/GeotechnicalDesignandDamSafety.aspx

A-7.5 SUPPLEMENTAL EXHIBITS

EXHIBIT A-7.1

General Quantities Calculations

Quantities for CID-KS

Demolition

Utility Demolition

See Utility Spreadsheet for details

Access and Staging Areas		
Staging Area 1		
Clearing and Grubbing Area at 6-inch depth	1.7	Acres
Staging Area 4		
Clearing and Grubbing Area at 6-inch depth	0.3	Acres
Fence Lines		
Removal of fence line at Area fill 32+50 to 38+00		
Length	900	ft
Removal of fence line at Area fill 77+00 to 94+50		
Length	400	ft
Pavements and lighting		
Removal of Asphalt at Area Fill 32+50 to 38+00		
Area	171919	ft^2
Number of light poles (wood poles)	8	each
Removal of Asphalt, curb & gutter at Area Fill 77+00 to 94+50		
Area	176680	ft^2
Number of light poles (Aluminum poles)	8	each
Curb and gutter	3300	
Other		
High Mast Light Pole	1	unit
Billboard	1	unit

Construction

Utility Relocation

See Utility Spreadsheet for details

Access and Staging Areas

Staging Area 1		
Length of chainlink fence	1579	ft
Number of gates	1	
Total Area	72301	ft^2
Depth of Aggregate Surface Course	6	inches
Volume ot Aggregate Surface Course	1339	Yd^3
Tons of Aggregate Surface Course (1.8 Tons/Yd^3)	2410	Ton
Staging Area 2		
Length of chainlink fence	1262	ft
Number of gates	2	
Total Area	81790	ft^2
Depth of Aggregate Surface Course	6	inches
Volume ot Aggregate Surface Conrse	1515	Yd^3
Tons of Aggregate Surface Course (1.8 Tons/Yd^3)	2726	Ton
Staging Area 3		
Length of chainlink fence	1218.56576	ft
Number of gates	2	
Total Area	57681.2511	ft^2
Depth of Aggregate Surface Course	6	inches
Volume ot Aggregate Surface Course	1068	Yd^3
Tons of Aggregate Surface Course (1.8 Tons/Yd^3)	1922.70837	Ton
Staging Area 4		
Length of chainlink fence	938	ft
Number of gates	1	
Total Area	43977.8327	ft^2
Depth of Aggregate Surface Course	6	inches
Volume ot Aggregate Surface Course	814.4	Yd^3
Tons of Aggregate Surface Course (1.8 Tons/Yd^3)	1465.9	Ton
Access Roads		
Total Area	167980	ft^2
Depth of Aggregate Surface Course	6	inches
Volume ot Aggregate Surface Course	3111	Yd^3
Tons of Aggregate Surface Course (1.8 Tons/Yd^3)	5599	Ton

Fence Lines

Replace fence line at Area fill 32+50 to 38+00		
Length	900	ft

Replace fence line at Area fill 77+00 to 94+50		
Length	400	ft
Pavements and lighting		
Replace Asphalt at Area Fill 32+50 to 38+00		
Area	171919	ft^2
Depth of Asphalt	5	inches
Volume of Asphalt	2653	Yd^3
Depth of Aggregate Surface Course	6	inches
Volume of Aggregate Surface Course	3184	Yd^3
Replace Number of light poles (wood poles)	8	each
Replace Asphalt, curb & gutter at Area Fill 77+00 to 94+50		
Area	176680	ft^2
Depth of Asphalt	5	inches
Volume of Asphalt	2727	Yd^3
Depth of Aggregate Surface Course	6	inches
Volume of Aggregate Surface Course	3272	Yd^3
Replace Number of light poles (Aluminum poles)	8	each
Other		
High Mast Light Pole	1	unit
Billboard	1	unit

Staging Area Deconstruction and Seeding and Mulching

Contractor shall remove fencing, remove aggregate surfacing, backfill with top soil, grade, and seed and mulch

Staging Area 1		
Length of chainlink fence REMOVAL	1579	ft
Number of gates REMOVAL	1	
Total Area	72301	ft^2
Depth of Aggregate Surface Course	6	inches
Volume ot Aggregate Surface Course	1339	Yd^3
Tons of Aggregate Surface Course (1.8 Tons/Yd^3) REMOVAL	2410	Ton
Seeding and Mulching	2	acres
Staging Area 2		
Length of chainlink fence REMOVAL	1262	ft
Number of gates REMOVAL	2	
Total Area	81790	ft^2
Depth of Aggregate Surface Course	6	inches
Volume ot Aggregate Surface Course	1515	Yd^3
Tons of Aggregate Surface Course (1.8 Tons/Yd^3) REMOVAL	2726	Ton
Seeding and Mulching	2	acres
Staging Area 3		
Length of chainlink fence REMOVAL	1218.56576	ft
Number of gates REMOVAL	2	
Total Area	57681.2511	ft^2
Depth of Aggregate Surface Course	6	inches
Volume ot Aggregate Surface Course	1068	Yd^3
Tons of Aggregate Surface Course (1.8 Tons/Yd^3) REMOVAL	1922.70837	Ton
Seeding and Mulching	1	acres
Staging Area 4		
Length of chainlink fence REMOVAL	938	ft
Number of gates REMOVAL	1	
Total Area	43977.8327	ft^2
Depth of Aggregate Surface Course	6	inches
Volume ot Aggregate Surface Course	814.4	Yd^3
Tons of Aggregate Surface Course (1.8 Tons/Yd^3) REMOVAL	1465.9	Ton
Seeding and Mulching	1	acres

Seeding and Mulching at borrow area.

Maximum area to be reseeded	88	acres

Seeding and Mulching on the levee and area fills

Length of levee	6515	ft
Length of slope on levee	775	ft
Area of levee to be seed and mulched	116	acres
Area fill 35+50 to 38+00	0	
Area fill 63+00 to 74+75	1.7	acres
Area fill 63+00 to 74+75	6.6	acres

EXHIBIT A-7.2

Borrow Area for Proposed Argentine, Armourdale, and CID-KS Unit Raise

Borrow Area for Proposed Argentine, Armourdale, and CID-KS Unit Raise

Prospective borrow areas were identified by the Sponsor and screened through joint Corps and Sponsor efforts during the Phase I portion of this feasibility project. Regarding borrow area, Phase 1 focused on the Argentine Unit. Because the borrow area for Armourdale will be the same as for Argentine, this write-up was taken from the Phase I report and updated to include the Armourdale Unit raises.

ARGENTINE

Total required fill quantities for Argentine are 90,301, 257,881, and 508,281 compacted cubic yards (ccy) for N500, N500+3, and N500+5 raises, respectively. The N500+3 raise is the recommended alternative for Phase 1, therefore 258,000 ccy will be used for estimating borrow needs. For Argentine, the proposed levee raise accounts for about half of the fill requirement and stability or underseepage berms account for the other half. Subsurface investigation of the borrow area provided the required geotechnical information for the materials to be used in the levee.

ARMOURDALE

Total required borrow quantities from WaterOne for Armourdale is approximately 330,000 bank cubic yards (bcy) for the N500+3 raise. This quantity is based on balancing cut and fill on site and then calculating what is required to supplement on-site material with that from WaterOne.

CID-KS

Total required borrow quantities for WaterOne for CID-KS is approximately 88,300 bcy for the N500+3 raise. This quantity is based on balancing cut and fill on site and then calculating what is required to supplement on-site material with that from WaterOne.

The attached WaterOne Borrow Area Typical Cross Section and table summarize the borrow material needs for Argentine, Armourdale and CID-KS. See "WaterOne Borrow Area Typical Cross Section", for a typical cross section for the borrow area excavation.

Roughly 45 acres will be needed for Armourdale's 330,000 bank cubic yard requirement and roughly 18 acres will be needed for CID-KS's 88,300 bank cubic yard requirement. This assumes that impervious material is obtained in a 3-foot layer with random material obtained below that from a 2 to 3-foot layer. For all of borrow requirements for Argentine, Armourdale, and CID-KS, approximately 93 acres will be needed assuming impervious material is obtained from a 3-foot layer and random obtained from a 2 to 3 foot layer below that. The impervious layer thickness assumption is critical to estimating the acreages to be used from WaterOne. The 3-foot impervious thickness assumption was derived from the eight boring logs.

Cultural resource investigation into the WaterOne borrow site resulted in a maximum excavation depth of 10 feet. The US Army Corps of Engineers recommended a maximum excavation depth of 10 feet and the Kansas State Historic Preservation Officer has concurred with Corps' recommendation that no archeological survey is needed for borrow activity kept to a depth of 10 feet or less.

BORROW AREA SEARCH

Originally, the Argentine & Armourdale foreshore areas were considered due to their close proximity to the Argentine unit. As HTRW investigations were undertaken for areas of interest, however, various regions of contamination were discovered which eliminated most of these areas from consideration. Total remaining available fill in these areas, ASSUMING NO FURTHER HTRW DISCOVERIES, is approximately 143,000 CY (see FIGURE 1 - "FORESHORE"). Figure 1 reflects avoidance of known HTRW concerns, a minimum 300' standoff distance from existing levees or floodwalls, and maximum depth of excavation of ordinary high water (OHW) minus 4 feet. It is recommended that this area be retained for further consideration during project engineering & design, though there is a possibility that further HTRW investigations will make even the remaining material unusable. Even if no further HTRW issues are discovered, any borrow from this area would need chemical analysis sampling at a rate of 1 sample (about $1000) per 5000 cy of borrow due to the known contamination and associated legal entanglements in the area.

Since the remaining foreshore quantity alone (assuming future HTRW clearance) is marginal for the N500 raise and insufficient for the other two prospective modifications to the Argentine unit, efforts were taken to identify alternative borrow areas as close to the project as possible. FIGURE 2 - "VICINITY MAP" shows various sites considered and investigated. Many of the sites near the project area were either very small or had other undesirable characteristics such as extremely high land values or prior industrial use. Several areas, as discussed on the following pages, were further investigated.

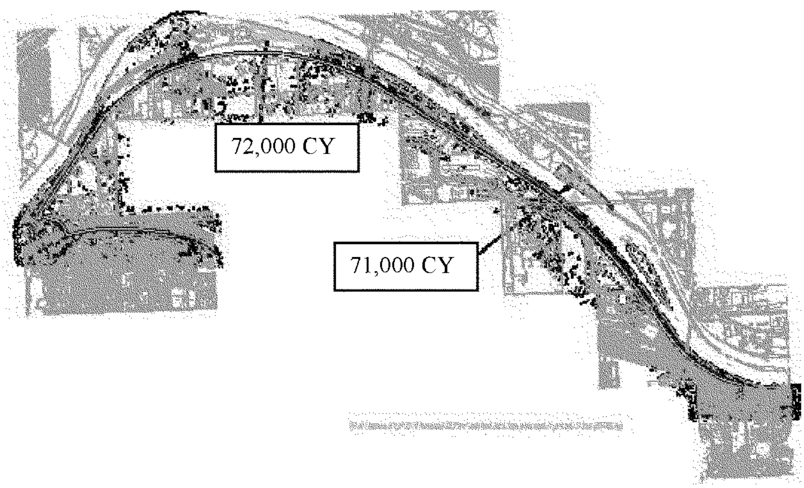


FIGURE 1 – FORESHORE

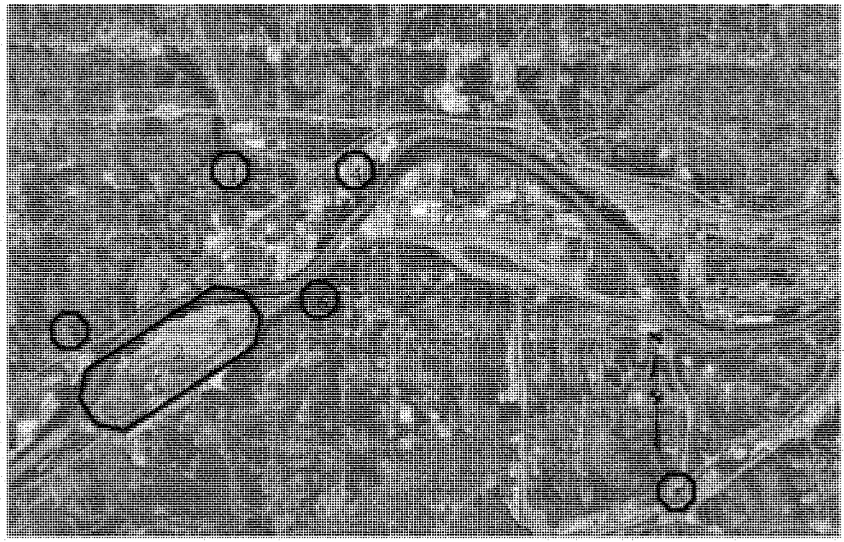


FIGURE - 2 VICINITY MAP

Area 1 is an open field south of the Turner Diagonal. The area is approximately 40 acres and appears to have been previously used as a borrow area. Since access to and from the area requires travel through residential neighborhoods on narrow routes, the area was not considered for further study.

Area 2 is in the Kansas River floodplain. The area is approximately 500 acres, 380 of which are owned by Water District One of Johnson County (WaterOne). This area appeared to be a good candidate for further consideration, and is discussed in detail below.

Area 3 is owned by Amino Brothers Construction and has previously been used as source of borrows. Approximately 200,000 cy of material is available, per conversation with the owner. This site may be a viable backup source for impervious materials, if required.

Area 4 is approximately 50 acres and used for a variety of commercial / industrial purposes. Since current appraised land values are in excess of $2000 per acre, this area was not considered for further study.

Area 5 is owned by Sandifer Leasing and has previously been used as source of borrows. Field investigations show little to no remaining fill, therefore the site was not considered for further study.

Area 6 is a large wooded hillside, which appears to be undisturbed. The area below is covered by a network of tunnels, originally used for limestone mining and currently for cold storage. Due to the likelihood of disturbing the tunnels below during earth moving operations, this area was not considered for further study.

See TABLE 1 "BORROW AREA COMPARISON" for a summary comparison of prospective sites.

TABLE 1 - BORROW AREA COMPARISON

AREA	OWNER	HAUL DIST	PROS	CONS	ACTION
1	Unknown	2 miles	Close to site	Residential access, small	Remove from consideration
2	WaterOne	4 miles	Little or no cost	Haul distance	Investigate as primary source
3	Amino Bros.	5 miles	Bank source – expected to be impervious	Haul distance, cost of fill	Keep for possible contingency
4	5701 LLC	2 miles	Close to site	High cost of comm/ind property, developed	Remove from consideration
5	Sandifer	5 miles	None	Haul distance, look like no fill left	Remove from consideration
6	Unkonwn	3 miles	Bank source- expected impervious	Haul distance, likelihood of damaging tunnels below	Remove from consideration
Foreshore	KVDD easement	0 (Argentine) 4 miles (Armourdale)	Very close to site	Potential HTRW, legal entanglements, high chemical sampling cost	Keep for possible contingency

Area 2, shown below in additional detail in FIGURE 3, contains approximately 500 acres and is bounded by the Kansas River and Holliday Drive. WaterOne owns 380 acres in this area and uses the site for disposal of quicklime used in the water treatment process. Individual cells, each 5-10 acres and 20 feet deep, are excavated and, over the course of 3-5 years, filled with dewatered lime (40-60% solids). The cells are then capped with soil, and the excess soil stockpiled elsewhere onsite. During an October 2004 meeting with WaterOne staff, the requirements for the Argentine levee raise project were discussed in detail. WaterOne staff indicated a desire to dispose of excess materials and was interested in pursuing an agreement for use of the excess materials. Soil boring logs for previous WaterOne well and disposal cell construction indicate significant deposits of silt and silty clay, both of which would qualify as impervious fill, in the area.

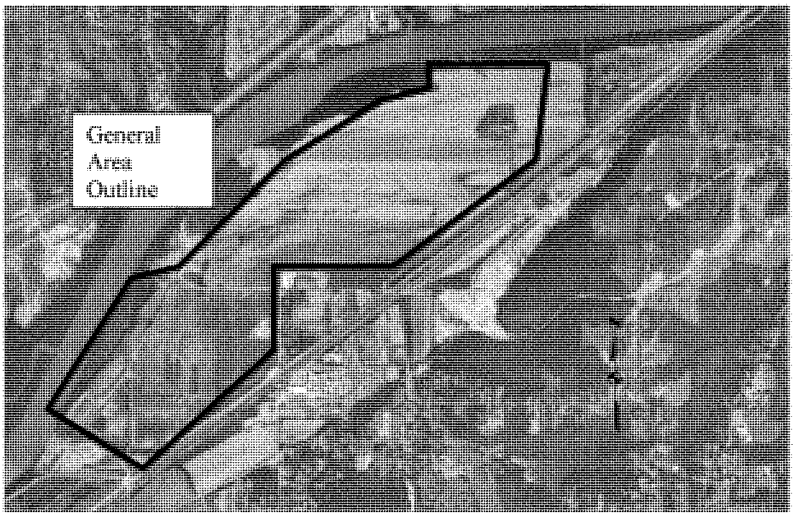


FIGURE 3 – BORROW AREA 2 - WATER ONE

Exploratory soil borings and chemical analysis sampling was conducted in January 2005. Chemical analysis entailed 3 grab samples for volatile organic compounds (VOCs) and three composite samples for metals, pesticides herbicides, and semivolatile organic compounds (SVOCs). Chemical analysis sampling points differed from soil boring locations, but were taken at various locations throughout the WaterOne property to assure representative results. All parameters tested were below action levels.

Subsurface Investigation.

Exploratory Borings.

The subsurface investigation of the borrow area consisted of 8 exploratory borings, 10 -feet deep, drilled with 3 ¾ ID Hollow Stem Auger with 3-inch inner barrel sampler. The borings location with WaterOne property delineated is shown in FIGURE 4 and the strip logs are included at the end of the paragraph. All holes were backfilled prior to leaving the site. No water was encountered during drilling or 24 hours after drilling. Forty (40) jar samples and 8 sack samples (1 composite sack sample for each boring) were collected from all borings. The boring logs show an impervious soil layer consisting of silts and clays extending up to 6 feet below the surface followed by a sandy aquifer. The central part of the borrow area has a thin layer of sand at the surface, varying between 1 and 4.5 feet in thickness, followed by 3 to 4 feet of silts and clay, on the top of the sandy aquifer. The sandy material can be used as backfill in the random portion of the landside levee embankment.

Laboratory Testing.

Selected samples of material obtained during the field exploration were tested to determine engineering and physical properties of the soils. Laboratory testing was performed by Geotechnology, Inc. The laboratory testing included Atterberg Limits, natural moisture contents, and Standard Proctor tests. The samples were grouped in 5 categories of similar characteristics and Atterberg Limits were performed on a representative sample of each category. The moisture content varies between 4 and 35%. Overburden clay and silt material was classified in accordance with ASTM D 2487 as lean clay (CL) or silt (ML). Three of the groups were determined to be non plastic, the other 2 groups were classified one as a lean clay (CL) and the other as silt (ML). The silt was determined to be non-plastic material. The Liquid Limit (LL) of the CL material varies between 39 and 47 and the Plasticity Index (PI) between 19 and 28. The results of the natural moisture content tests and performed on twenty five (25) disturbed samples and of the Atterberg Limits tests performed on 2 selected representative samples of clay material are shown in an enclosure at the end of the paragraph.

Three Standard Proctor Tests were performed on composite samples collected from the borrow areas conform ASTM D-698. The materials were classified as low plasticity clay with the LL between 52 and 55 and PI between 35 and 37 respectively. The maximum dry density varied between 107.5 and 102 pcf with the optimum moisture content varying between 18.5% and 20.5%.

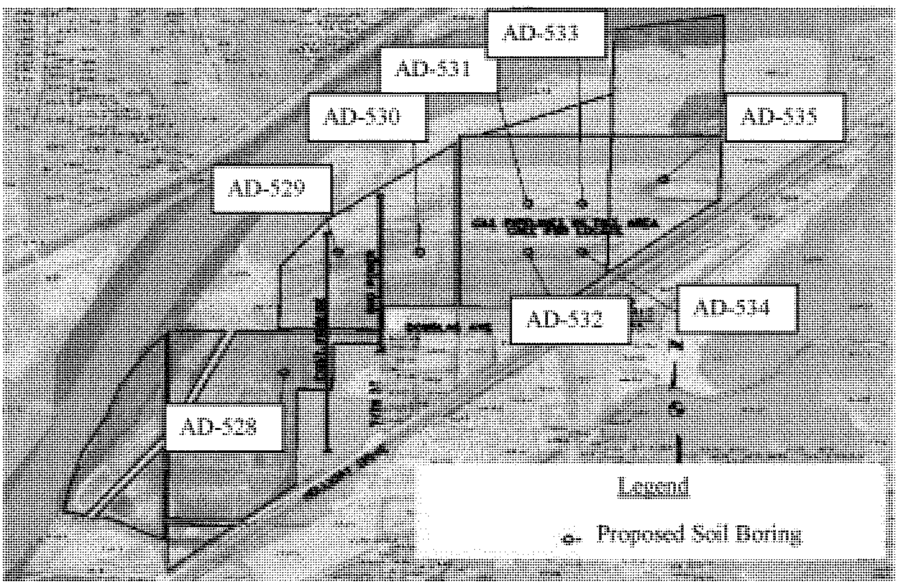
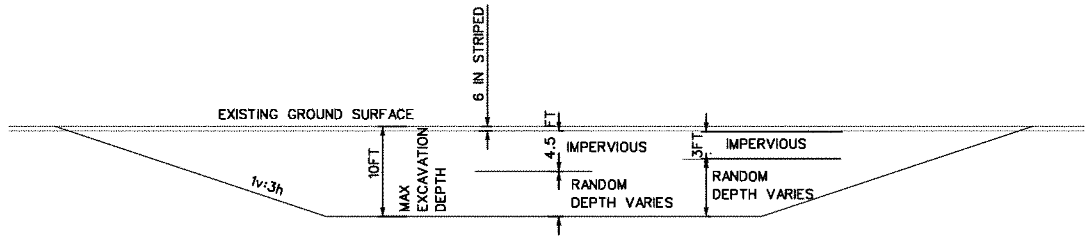


FIGURE 4 - SOIL BORINGS



Borrow Area Needs - Current Recommendation N500+3
Using Two Depths (3' and 5') for Impervious Excavation

					Impervious		Total		
	Total Borrow Quantity	Impervious	Random	Strip Top 6"	Depth of Excavation for Impervious Only	Area of Excavation	Maximum Depth of Excavation (1)	Minimum Depth of Excavation (4)	Total Area of Excavation (5)
	BCY	BCY	BCY		Feet	Acres	Feet	Feet	Acres
Armourdale	330,000	220,000	110,000	0.5	3	45	10	5	45
	330,000	220,000	110,000	0.5	5.0	27	10	8	27
CID-KS	88,300	88,260	-	0.5	3	18	10	4	18
	88,300	88,260	-	0.5	5.0	11	10	6	11
Argentine	282,700	144,226	138,490	0.5	3	30	10	6	30
	282,700	144,226	138,490	0.5	5.0	18	10	10	18
Total	701,000	452,486	248,490	0.5	3	93	10	5	93
	701,000	452,486	248,490	0.5	5.0	56	10	8	56

Notes
CCY Compacted Cubic Yards
BCY Bank Cubic Yards = CCY/0.8
BOTEA Back of the Envelope Analysis
Assumptions: Impervious material is generally located above other material
Estimate will be revised as levee/floodwall alignment is refined.
(1) Maximum depth of excavation is set to 10 feet.
Kansas State Historic Preservation Officer has concurred with Corps' recommendation that no archeological survey is needed for borrow activity kept to a depth of 10' or less
(2) (3) Comments Not used
(4) Minimum excavation depth is calculated using Alternative 2 - setting the acreage needed for random fill equal to that needed for impervious fill.
(5) Total area of excavation is the greater value returned when comparing the Impervious Area of Excavation and the Random Alt. 1 Area of Excavation

KANSAS CITYS LEVEES
FEASIBILITY PHASE 2

WATERONE
BORROW AREA
TYPICAL
CROSS SECTION

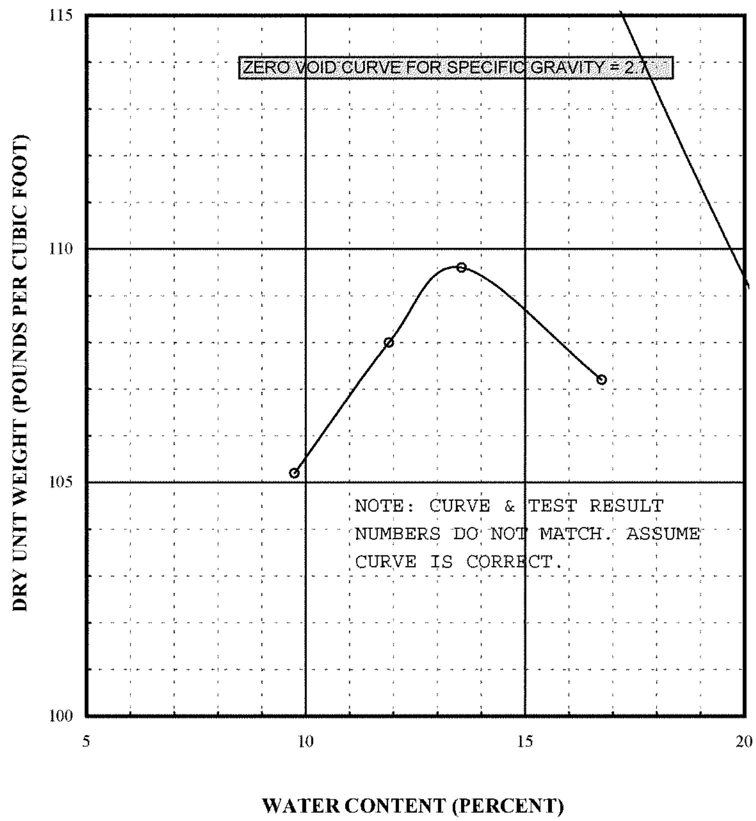
Date: November 13, 2008

	BORING NO.	Sample Depth (feet)	Sample No.	Group Class.	LABORATORY TESTS: Moisture Content % (ASTM D2216)	Atterberg Limits (ASTM D4318): Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Standard Proctor (ASTM 698 Method A): Max. Dry Density	Standard Proctor: Optima Water Content	Classification or Group Classification ASTM D2487
SACK SAMPLES	AD-528	0.8-4.0	Sack-1	--	--	29	21	8	104.7	18.2	CL-dark brown sandy low plasticity CLAY
SACK SAMPLES	AD-529	3.0-6.0	Sack-1	--	--	27	17	10	113.9	14.6	SC-dark brown clayey SAND
SACK SAMPLES	AD-535	1.0-4.5	Sack-1	--	--	47	19	28	102.5	19.0	CL-dark brown sandy low plasticity CLAY
DISTURBED SAMPLES	AD-528	0.0-0.5	J-1	5	29.2	Not enough sample					
		0.5-0.8	J-2	5							
		0.8-4.0	J-3	1							
		4.0-4.6	J-4	1	25.0	39	20	19			
		4.6-8.4	J-5	3							
		8.4-9.0	J-6	2							
	AD-529	0.0-1.0	J-1	4	27.5						
		1.0-3.0	J-2	3							
		3.0-4.0	J-3	4	21.2						
		4.5-6.0	J-4	1	24.9						
		6.0-10.0	J-5	3							
	AD-530	0.0-1.0	J-1	4							
		1.0-1.5	J-2	3	4.0						
		1.5-2.5	J-3	2	20.1	Non-plastic					
		2.5-4.3	J-4	2	11.3						
		4.3-8.0	J-5	2	25.2						
		8.0-9.3	J-6	4		Non-plastic					

Group Classification Number:

1. CL - dark brown sandy low plasticity CLAY
2. ML - light brown low plasticity SILT
3. SP - tan fine-grained SAND
4. SM - dark brown silty SAND
5. FILL - dark brown gravelly low plasticity CLAY with sand
6. CL - dark brown low plasticity CLAY

BORING NO.	Sample Depth (feet)	Sample No.	Group Class.	LABORATORY TESTS: Moisture Content %	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Standard Proctor: Max. Dry Density	Optima Water Content	Classification or Group Classification ASTM D2487
				ASTM D2216	ASTM D4318			ASTM 698 Method A		
AD-531	0.0-1.0	J-1	4	14.4						
	1.0-3.5	J-2	1	22.2						
	3.5-4.0	J-3	2							
	4.0-6.5	J-4	2	25.1						
	6.5-9.0	J-5	3		Non-plastic					
AD-532	0.0-4.3	J-1	1	26.3						
	4.3-6.3	J-2	1	30.7						
	6.3-8.3	J-3	2							
AD-533	0.0-2.3	J-1	1	18.9						
	2.3-4.3	J-2	2	9.9						
	4.3-6.0	J-3	2	14.9						
	6.0-8.0	J-4	1	26.2						
	8.0-9.3	J-5	2							
ADU-534	0.0-4.0	J-1	1	25.7						
	4.0-6.0	J-2	2	30.7						
	6.0-6.5	J-3	6	35.4						
	6.5-7.0	J-4	1	28.3						
	7.0-9.0	J-5	2	15.5						
AD-535	0.0-4.5	J-1	6	20.3	47	19	28			
	4.5-7.5	J-2	1	29.5						
	7.5-8.5	J-3	2							
	8.5-9.5	J-4	3							
	9.5-10.0	J-5	2							



PROJECT NAME

Argentine Levee Unit – Borrow Area

SPECIFICATIONS

Standard Proctor
ASTM D 698 Method A
Percent of Compaction N/A
Moisture Range ±% N/A

PROCTOR TEST RESULTS

Max. Dry Density	Optimum Water Content
104.7 pcf	18.2%

ATTERBERG LIMITS (ASTM D-4318)

Liquid Limit	Plastic Limit	Plasticity Index
29	21	8

DESCRIPTION

Dark brown, sandy low plasticity CLAY

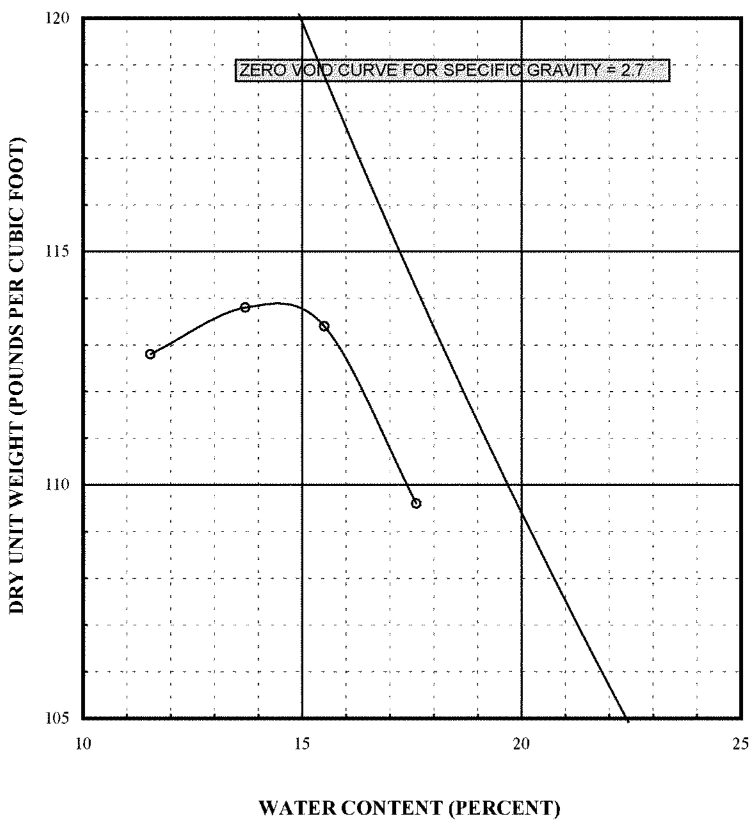
SAMPLE LOCATION

AD 528, 0.8-4.0 feet below grade

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
St. Louis, Collinsville, Kansas City

COMPACTION TEST

Job No.	0680806.52KM	**Test Date**	3/4/2005
Sampled By	USCOE	**Tested By**	SD
Sample Date	1/18/2005	**Calc. By**	YAW
Proctor No.	1075	**Ch'd By**	ARK



PROJECT NAME

Argentine Levee Unit – Borrow Area

SPECIFICATIONS

Standard Proctor
ASTM D 698 Method A
Percent of Compaction N/A
Moisture Range ±% N/A

PROCTOR TEST RESULTS

Max. Dry Density	Optimum Water Content
113.9 pcf	14.6%

ATTERBERG LIMITS (ASTM D-4318)

Liquid Limit	Plastic Limit	Plasticity Index
27	17	10

DESCRIPTION

Dark brown clayey SAND

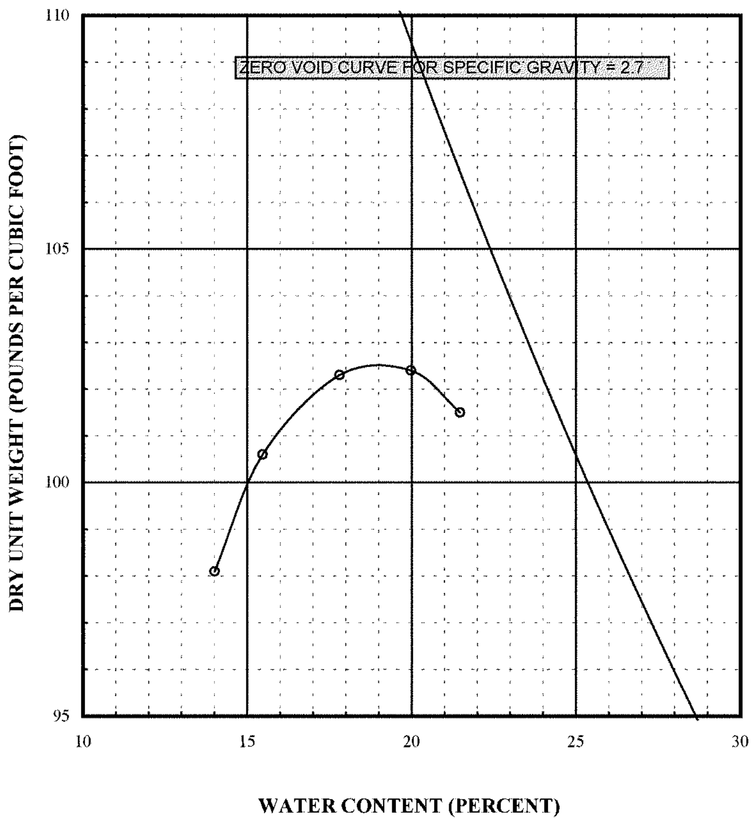
SAMPLE LOCATION

AD 529, 3.0-6.0 feet below grade

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
St. Louis, Collinsville, Kansas City

COMPACTION TEST

Job No.	0681901.32KT	**Test Date**	3/7/2005
Sampled By	USCOE	**Tested By**	SD
Sample Date	1/18/2005	**Calc. By**	YAW
Proctor No.	1077	**Ch'd By**	ARK



PROJECT NAME

Argentine Levee Unit – Borrow Area

SPECIFICATIONS

Standard Proctor
ASTM D 698 Method A
Percent of Compaction N/A
Moisture Range ±% N/A

PROCTOR TEST RESULTS

Max. Dry Density	Optimum Water Content
102.5 pcf	19.0%

ATTERBERG LIMITS (ASTM D-4318)

Liquid Limit	Plastic Limit	Plasticity Index
47	19	28

DESCRIPTION

Dark brown sandy low plasticity CLAY

SAMPLE LOCATION

AD 535, , 1.0-4.5 feet below grade

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
St. Louis, Collinsville, Kansas City

COMPACTION TEST

Job No.	0681901.32KT	**Test Date**	3/4/2005
Sampled By	USCOE	**Tested By**	SD
Sample Date	1/18/2005	**Calc. By**	YAW
Proctor No.	1076	**Ch'd By**	ARK

LOG OF BORING AD-528

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-528
LOCATION: Kansas and Missouri
COORDINATES: N 14190735.79, E 1141877.29 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driler: Mike Cooney Geologist:Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

FIELD DATA columns: DEPTH (ft); SOIL SYMBOL; BREAKS: bb or mb; SAMPLE/DRILL METHOD; BLOWS; T: TORVANE KG/CM SQ; RC: %, RQD: %, Additional Field Data

LABORATORY DATA columns: USCS SYMBOL; ATTERBERG LIMITS (LIQUID LIMIT LL, PLASTIC INDEX PI); MOISTURE CONTENT (%); VG=Visual Grouping, FC=Field Classification; OTHER LAB DATA (S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH)

DEPTH (ft)	DESCRIPTION OF STRATUM	Bottom depth	USCS SYMBOL	LL	PI	MOISTURE CONTENT (%)	VG/FC	OTHER LAB DATA
0	CLAY HARD DRY BROWN frozen	0.5				29.2	VG5	
	GRAVELLY COBBLES MODERATELY HARD GREY decomposed rock	0.8					VG5	
	LEAN CLAY MEDIUM DAMP - MOIST DARK BROWN	4.6	CL	29	8		VG1	
4			CL	39	19	25	VG1	
	FINE SAND LOOSE - MEDIUM COMPACT DRY LIGHT BROWN	8.4					VG3	
	SILT LOOSE DRY GREY	10.0					VG2	
10	Bottom of hole - No Refusal Backfilled to surface with cuttings and 3 bags Holeplug							

LEGEND:
- Fill (made ground)
- USCS Low Plasticity Clay
- USCS Poorly-graded Sand
- USCS Silt

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML; VG3 - SP; VG5 - FILL

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-529

SHEET 1 of 1

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-529
LOCATION: Kansas and Missouri
COORDINATES: N 14192413.68, E 1142543.89 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney Geologist: Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

Depth (ft)	Description of Stratum	Bottom (ft)	USCS Symbol	LL	PI	Moisture Content (%)	VG/FC	Other Lab Data
0	FINE SAND FROZEN DARK BROWN	1.0				27.5	VG4	
	FINE SAND LOOSE DRY BROWN	3.0					VG3	
	CLAYEY SAND MEDIUM COMPACT DAMP-MOIST DARK BROWN	4.5	SC	27	10	21.2	VG4	
	CLAY SOFT DAMP DARK BROWN very silty	6.0				24.9	VG1	
	FINE SAND LOOSE-MEDIUM DRY-DAMP LIGHT BROWN silty	10.0					VG3	
10	Bottom of hole - No Refusal. Backfilled to surface with cuttings and 3 bags Holeplug							

Column headings: FIELD DATA: DEPTH (ft); SOIL SYMBOL; BREAKS: bb or mb; SAMPLE/DRILL METHOD; BLOWS; T: TORVANE KG/CM SQ; RC: %, RQD: %, Additional Field Data. LABORATORY DATA: USCS SYMBOL; ATTERBERG LIMITS: LIQUID LIMIT (LL), PLASTIC INDEX (PI); MOISTURE CONTENT (%); VG=Visual Grouping, FC=Field Classification; OTHER LAB DATA.

OTHER LAB DATA: S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH

LEGEND:
USCS Silty Sand
USCS Poorly-graded Sand
USCS Clayey Sand
USCS Low Plasticity Clay

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG3 - SP; VG4 - SM

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-530

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-530
LOCATION: Kansas and Missouri
COORDINATES: N 14192416.14, E 1143534.5 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/12/05 - 1/18/05

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney Geologist: Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

FIELD DATA: DEPTH (ft); SOIL SYMBOL; BREAKS: bb or mb; SAMPLE/DRILL METHOD; BLOWS; T: TORVANE KG/CM SQ; RC: %, RQD: %, Additional Field Data

LABORATORY DATA: USCS SYMBOL; ATTERBERG LIMITS (LIQUID LIMIT LL, PLASTIC INDEX PI); MOISTURE CONTENT (%); VG=Visual Grouping, FC=Field Classification; OTHER LAB DATA (S: Minus 200 Sieve (%), U: Unconfined Compressive Strength (tsf), C: Confining Pressure (psi), F: Failure Strain (%), T: Total Sulfates, P: Soil pH)

Depth (ft)	Description of Stratum	Bottom depth	USCS Symbol	LL	PI	Moisture Content (%)	VG/FC	Other Lab Data
0	SILTY SAND, FROZEN, DARK BROWN, fine grained	1.0					VG4	
1.0		1.5				4	VG3	
1.5	FINE SAND, LOOSE, DRY-DAMP, BROWN, poorly graded	2.5				20	VG2	
2.5	SILT, MEDIUM COMPACT, DAMP, DARK BROWN	4.3				11	VG2	
4.3	SILT, MEDIUM COMPACT, LIGHT BROWN; SILT, MEDIUM COMPACT, DAMP, GRAYISH BROWN, sandy, wet zone	8.0				25	VG2	
8.0	SILTY SAND, MEDIUM COMPACT, DAMP, LIGHT BROWN, laminated, fine grained	10.0					VG4	
10.0	Bottom of hole - No Refusal. Backfilled to surface with cuttings and 3 bags Holeplug							

LEGEND: USCS Silty Sand; USCS Poorly-graded Sand; USCS Silt

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG2 - ML; VG3 - SP; VG4 - SM

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-531

SHEET 1 of 1

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-531
LOCATION: Kansas and Missouri
COORDINATES: N 14193052.59, E 1144847.77 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

FIELD DATA

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney   Geologist: Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling   ▼ Water level after drilling

LABORATORY DATA

OTHER LAB DATA: S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH

Field data columns: DEPTH (ft); SOIL SYMBOL; BREAKS: bb or mb; SAMPLE/DRILL METHOD; BLOWS; T: TORVANE KG/CM SQ; RC: %, RQD: %, Additional Field Data

DEPTH (ft)	DESCRIPTION OF STRATUM	Bottom depth	USCS SYMBOL	LL	PI	MOISTURE CONTENT (%)	VG=Visual Grouping / FC=Field Classification	OTHER LAB DATA
0	SILTY SAND, FROZEN, LIGHT BROWN, very fine grained	1.0				14	VG4	
	LEAN CLAY, SOFT-VERY SOFT, DAMP, DARK BROWN, very silty ~ 30-40 % silt	3.5				22	VG1	
	SILT	4.0					VG2	
	LOOSE, DRY, BROWN, with fine sand					25	VG2	
	SILT, MEDIUM COMPACT, DAMP, BROWN, slightly sandy ~ 10-15 % very fine sand	6.5						
	FINE SAND, MEDIUM COMPACT - LOOSE, DAMP-DRY, LIGHT BROWN	10.0					VG3	

Bottom of hole - No Refusal
Backfilled to surface with cuttings and 3 bags Holeplug

LEGEND: USCS Silty Sand; USCS Low Plasticity Clay; USCS Silt; USCS Poorly-graded Sand

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML; VG3 - SP; VG4 - SM

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-532

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-532
LOCATION: Kansas and Missouri
COORDINATES: N 14192422.33, E 1144971.11 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney   Geologist:Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling   ▼ Water level after drilling

FIELD DATA: DEPTH (ft), SOIL SYMBOL, BREAKS: bb or mb, SAMPLE/DRILL METHOD, BLOWS, T: TORVANE KG/CM SQ, RC: %, RQD: %, Additional Field Data

LABORATORY DATA

OTHER LAB DATA: S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH

DEPTH (ft)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL	LIQUID LIMIT LL	PLASTIC INDEX PI	MOISTURE CONTENT (%)	VG=Visual Grouping FC=Field Classification	OTHER LAB DATA
0 – 4.3	LEAN CLAY MEDIUM MOIST DARK BROWN frozen to 1.0 ft					26	VG1	
4.3 – 6.3	LEAN CLAY MEDIUM MOIST-WET DARK BROWN silty					31	VG1	
6.3 – 10.0	SILT MEDIUM COMPACT DRY-DAMP LIGHT BROWN sandy						VG2	
10.0	Bottom of hole - No Refusal Backfilled to surface with cuttings and 3 bags Holeplug							

LEGEND: USCS Low Plasticity Clay; USCS Silt

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-533

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-533
LOCATION: Kansas and Missouri
COORDINATES: N 14193066.09, E 1145518.24 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

FIELD DATA

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney Geologist: Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

LABORATORY DATA

Field data columns: DEPTH (ft); SOIL SYMBOL; BREAKS: bb or mb; SAMPLE/DRILL METHOD; BLOWS; T: TORVANE KG/CM SQ; RC: %, RQD: %, Additional Field Data

Other lab data: S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH

DEPTH (ft)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL	LIQUID LIMIT (LL)	PLASTIC INDEX (PI)	MOISTURE CONTENT (%)	VG=Visual Grouping FC=Field Classification	OTHER LAB DATA
0–2.3	LEAN CLAY, MEDIUM, DAMP, DARK BROWN, silty					19	VG1	
2.3–4.3	SILT, MEDIUM COMPACT, DRY-DAMP, LIGHT BROWN, with fine-grained sand					10	VG2	
4.3–6.0	SILT, MEDIUM COMPACT, DRY-DAMP, BROWN, with very fine-grained sand					15	VG2	
6.0–8.0	LEAN CLAY, MEDIUM, MOIST, DARK BROWN, with silt					26	VG1	
8.0–10.0	SILT, MEDIUM COMPACT, DRY-DAMP, LIGHT BROWN, with very fine-grained sand						VG2	
10.0	Bottom of hole - No Refusal. Backfilled to surface with cuttings and 3 bags Holeplug							

Legend: USCS Low Plasticity Clay; USCS Silt

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-534

SHEET 1 of 1

US Army Corps of Engineers®

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-534
LOCATION: Kansas and Missouri
COORDINATES: N 14192405.24, E 1145517.16 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/19/05 - 1/19/05

FIELD DATA

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driller: Mike Cooney Geologist:Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

LABORATORY DATA

Field data columns: DEPTH (ft); SOIL SYMBOL; BREAKS: bb or mb; SAMPLE/DRILL METHOD; BLOWS; T: TORVANE KG/CM SQ; RC: %, RQD: %, Additional Field Data

OTHER LAB DATA: S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH

DEPTH (ft)	DESCRIPTION OF STRATUM	Bottom depth	USCS SYMBOL	LIQUID LIMIT (LL)	PLASTIC INDEX (PI)	MOISTURE CONTENT (%)	VG=Visual Grouping FC=Field Classification	OTHER LAB DATA
0	LEAN CLAY, MEDIUM, DAMP, DARK BROWN, silty	4.0				26	VG1	
4	SILT, MEDIUM COMPACT, WET, DARK BROWN, clayey	6.0				31	VG2	
6	LEAN CLAY, SOFT, MOIST-WET, DARK BROWN, silty	6.5				35	VG6	
6.5	LEAN CLAY, MEDIUM, MOIST-WET, DARK BROWN, silty	7.0				28	VG1	
7	SILT, MEDIUM COMPACT, DRY-DAMP, LIGHT BROWN, with very fine-grained sand	10.0				16	VG2	

Bottom of hole - No Refusal
Backfilled to surface with cuttings and 3 bags Holeplug

LEGEND
USCS Low Plasticity Clay
USCS Silt

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML; VG6 - CL(LL=47,PI=28)

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

LOG OF BORING AD-535

SHEET 1 of 1

US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Kansas City, Seven Levees
PROJECT: Argentine Levee Unit-Borrow Area
BORING NUMBER: AD-535
LOCATION: Kansas and Missouri
COORDINATES: N 14193402.93, E 1146504.01 ; NAD 83 UTM 15N feet
ELEVATION: 0.0 (ft)
DATE(S) DRILLED: 1/18/05 - 1/18/05

FIELD DATA: DEPTH (ft), SOIL SYMBOL, BREAKS: bb or mb, SAMPLE/DRILL METHOD, BLOWS, T: TORVANE KG/CM SQ, RC: %, RQD: %, Additional Field Data

DRILLING METHOD(S): Diedrich D-90, 3 3/4" ID hollow stem auger, 3" ID inner barrel sampler

Driler: Mike Cooney Geologist:Jennifer Denzer

GROUNDWATER INFORMATION:
No water encountered during drilling or after. Dry 1/19/05

▽ Water Level during drilling ▼ Water level after drilling

LABORATORY DATA

OTHER LAB DATA: S: Minus 200 Sieve (%); U: Unconfined Compressive Strength (tsf); C: Confining Pressure (psi); F: Failure Strain (%); T: Total Sulfates; P: Soil pH

DEPTH (ft)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL	LL	PI	MOISTURE CONTENT (%)	VG=Visual Grouping FC=Field Classification	OTHER LAB DATA
0–4.5	LEAN CLAY, SOFT, DAMP, DARK BROWN, silty ~ 10-15% silt		CL CL	47 47	28 28	20	VG6	
4.5–7.5	LEAN CLAY, MEDIUM, WET, DARK BROWN, silty					30	VG1	
7.5–8.5	SILT, MEDIUM COMPACT, DAMP, LIGHT BROWN, with very fine-grained sand						VG2	
8.5–9.5	FINE SAND, LOOSE, DRY, LIGHT BROWN, poorly graded						VG3	
9.5–10.0	SILT, MEDIUM COMPACT, DAMP-MOIST, BROWN, sandy						VG2	

Bottom of hole - No Refusal
Backfilled to surface with cuttings and 3 bags Holeplug

LEGEND: USCS Low Plasticity Clay; USCS Silt; USCS Poorly-graded Sand

R: BLOW COUNT REFUSAL = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
T - TORVANE EQUALLY SPACED ALONG SAMPLE
RC - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS: Coordinates Trimble Hand GPS
VG1 - CL(LL=39,PI=19); VG2 - ML; VG3 - SP; VG6 - CL(LL=47,PI=28)

LOG_A_2005 KANSAS-CITY-LEVEES.GPJ 4/1/05

Exhibit A-7.3

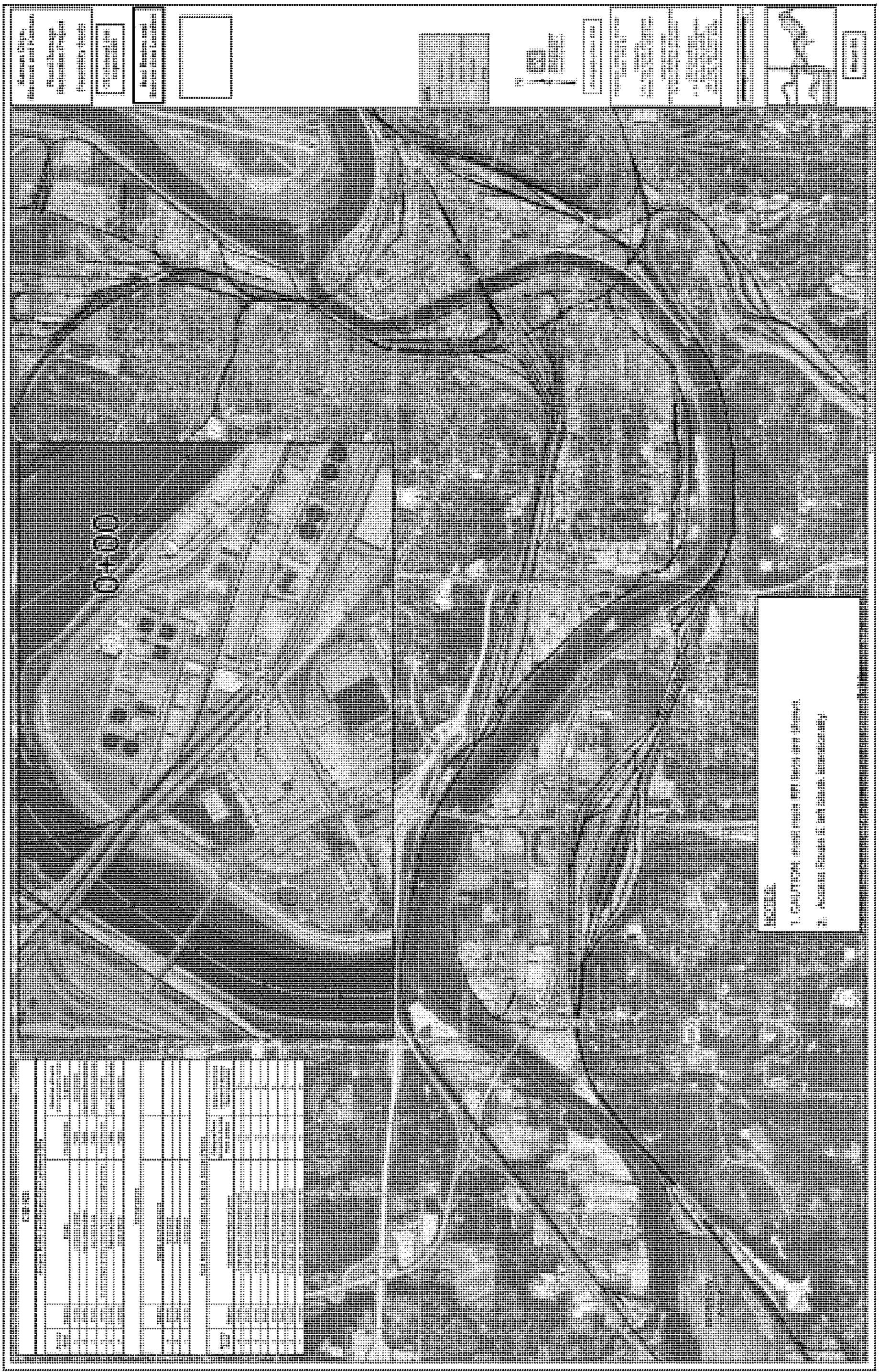


EXHIBIT A-7.4

Kansas City's Levee and Floodwall Gravity and Utility Pipeline Guidance

Reviewed: July 5, 2007

KANSAS CITY'S LEVEE AND FLOODWALL GRAVITY AND UTILITY PIPELINE GUIDANCE

PURPOSE

The purpose of this document is to provide specific guidance during the feasibility phase of the Kansas City's Levee project as to the disposition of existing utilities and drainage structures within the sections of levee and floodwall to be raised. This guidance will be used for the feasibility level of effort in order to develop reasonable costs associated with the modification of drainage structures and the relocation of utilities.

Uplift of utilities within the critical zone of the levee or floodwall will be addressed in accordance with COE criteria. Uplift is not addressed in this KCL guidance.

REFERENCES

	Local Protection – Web page guidance
	Local Protection - Guidebook on web page (Guidance for work Proposed Near or within Federally Constructed Flood Risk Management Projects)
EM 1110-2-1913	Design and Construction of Levees
EM 1110-2-2902	Conduits, Culverts, and Pipes
EM 1110-2-3102	General Principles of Pumping Station Design and Layout
EM 1110-2-3104	Structural and Architectural Design of Pumping Stations
EM 1110-2-3105	Mechanical and Electrical Design of Pumping Stations (Changes 1 of 2)

GRAVITY PIPELINES

Existing pipelines crossing the levee that do not meet current COE criteria shall be replaced with pipelines that are compliant. Existing pipelines that meet current COE criteria shall remain with the following exceptions:

Any Corrugated Metal Pipe (CMP) with a diameter greater than 36" shall be replaced with a minimum diameter 48" Reinforced Concrete Pipe (RCP).

Any pipe inadequate to handle the drainage shall be replaced with a minimum diameter 48" RCP.

Any pipe known to have joints that are not watertight shall be replaced with a minimum diameter 48" RCP.

For new pipe installations, CMP will not be allowed.

Pipe strengths, unless otherwise known, will be assumed to be that required by Corps criteria at the time of their installation. Pipe condition shall be determined by field assessment.

GATEWELLS AND POSITIVE CLOSURES

In areas where levee raises are performed, positive closure will be provided for all drainage and utility lines crossing the levee. EM 1110-2-1913 states that gravity lines that penetrate the embankment or foundation of a levee must be provided with devices to assure positive closure. This criteria also states that gravity lines should be provided with flap-type or slide-type service gates on the riverside of the levee. Because the KS River and MO River are not fast rising rivers, a flap gate will not be recommended on existing outfalls where sluice gates are present but no flap gate. For new outfall structures, however, flap gates will generally be installed.

Emergency means of closure is suggested for gravity lines in addition to the positive closure device. Historically, a flap gate on the end of the pipe has acted as this second closure device. However, it is possible to use sandbags or concrete to fill a gatewell as a means of emergency closure during a flood situation, although this is not the recommended alternative.

All gatewells within the Kansas City Levee study area are considered confined spaces. OSHA regulations and Corp EM 385-1-1 require anyone entering a confined space to comply with specific confined space entry requirements. New or modified gatewells will be designed so that these confined space entry requirements can be met. For example, space will be provided above the gatewell opening so that a tripod can be set to facilitate non-entry rescue.

NON-GRAVITY PIPELINES CROSSING THROUGH OR UNDER LEVEES

It is preferable for all pipes or conduits to cross over the levee rather than penetrate the embankment or foundation materials. This includes pipes carrying fiber optic, pressurized gas or pressurized liquid. Where raises are made to the levee, existing non-gravity pipelines should be relocated over the crest of the new levee raise. See detail "Typical Utility Crossing Levee Raise". A determination to relocate existing lines will be made on a case-by-case basis.

Pressure pipe

All pipes allowed to penetrate the embankment or foundation of a levee must be provided with devices to assure positive closure. These valves shall be placed in close proximity to the levee and have capability to be closed rapidly to prevent gas or fluid from escaping within or beneath a levee should the pipe rupture within these areas.

Provisions for closure of pressure pipes on the water side must also be provided to prevent backflow of floodwater into the protected area should the pipe rupture.

Casing Pipes, Cased Pipes and Conduits Crossing Through or Under Levees (Telecommunications)

It is preferred that conduits or casing pipes cross up and over the levee. However, where it is not possible to go over the levee, casing pipes or conduits must be installed in accordance with COE criteria. Refer to COE Guidebook located on the KC District website for directional drilling procedures.

ABANDONED PIPELINES

Pipelines, which are currently abandoned and grouted in accordance with COE criteria under or through the levee, will not be disturbed. Pipes that have been abandoned and do not meet criteria or it is unknown if they meet criteria shall be removed or properly abandoned according to COE criteria. Pipelines that are currently active but are to be abandoned as part of this project will be removed or abandoned according to COE criteria.

Removal

For feasibility purposes only, the following guidance is used in determining if an abandoned pipeline will be removed or abandoned in-place in accordance with Corps criteria.

> Where levee heights are less than 10 feet and when an abandoned utility is buried less than 5 feet below the base of the levee, the abandoned utility crossing under the levee should be removed unless special circumstances warrant a different approach.

Exploration Trench

For cost estimating purposes during feasibility, all known pipes are assumed to be located as shown on maps and plans or as located in the field during feasibility site visits.

No exploration trenches will be specified during feasibility. However, it is noted that during PED phase, it may be determined that exploration trenches will be needed during construction in order to find some utilities or to verify that some utilities do not exist as shown on the drawings.

Grouting Abandoned Pipelines

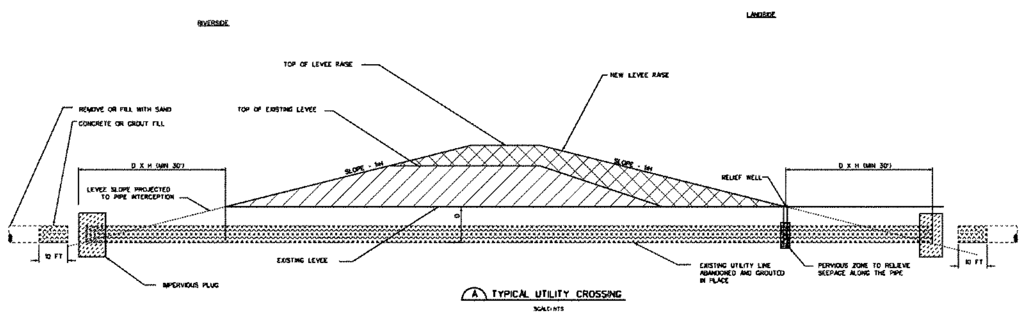
If a pipe does not meet the requirement for removal, the pipe should be properly abandoned by filling with a grout based substance, e.g., cement-bentonite, or flowable fill. The grout or flowable fill mix should be approved by the Corps of Engineers. The grout shall be fluid enough, and pumped in the up-slope direction so that the pipe will be completely filled leaving no voids. Points of access need to be made into the pipe at sufficient intervals to accomplish the grouting, see detail "Typical Utility Abandonment" for additional details regarding abandoning a utility in place.

OTHER CONSIDERATIONS

Special consideration will be given to existing pipe crossings on a case-by-case basis when hazardous, toxic, and radioactive waste (HTRW) concerns or real estate issues exist. HTRW concerns exist in various locations along the Kansas City Seven Levee system. When it is not desirable to disturb the existing ground due to HTRW concerns, the final recommendation for removing/relocating an existing utility will weigh the risks involved with disturbing the ground against leaving an existing utility in place. When real estate issues exist, the final recommendation will consider how real estate is affected.

SUMMARY OF RECOMMENDATIONS

For sections of levee or floodwall to be raised or modified, current Corps requirements will be extended to all components of that levee section, including any pipes and closure structures therein. When it is not practical to meet Corps requirement, each utility will be evaluated on a case-by-case basis.



A TYPICAL UTILITY CROSSING

SCALE: NTS

NOTES:
1. THE PREFERENCE IS TO REMOVE PIPES LOCATED IN OR UNDER THE LEVEE. IF NOT FEASIBLE, THIS DETAIL SHOULD BE ADHERED TO UNLESS APPROVED OTHERWISE. THIS DETAIL IS FEASIBILITY LEVEL DETAIL AND WILL REQUIRE ADDITIONAL INFORMATION FOR PED.

US Army Corps of Engineers
Kansas City District

Symbol	Description	Date	Appr.

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: MRC	Date: DECEMBER 2006
Drawn by: MRC	Drawing code: X
Checked by: HMW	Plot scale: NA
Submitted by: HMW	CADD File Name: UTILCROSSINGDETAIL.V8

KANSAS CITY LEVEES
FEASIBILITY STUDY PHASE 2
ARMOURDALE

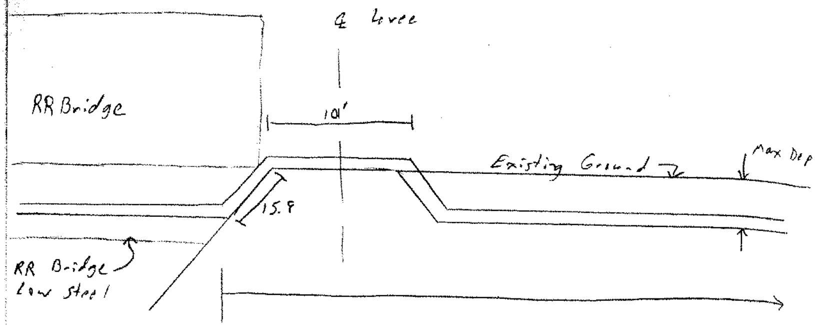
TYPICAL UTILITY
ABANDONMENT - LEFT IN PLACE

Sheet Reference Number:
XX

EXHIBIT A-7.5

Utilities that Require Relocation Calculations

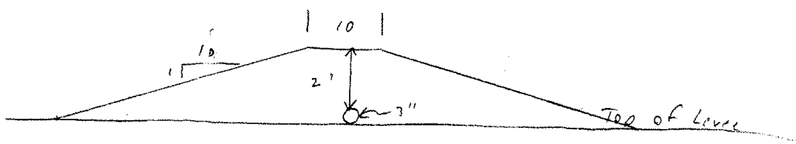
Fiber Optics at 18+10, 18+20, 18+25, 18+75, 19+85

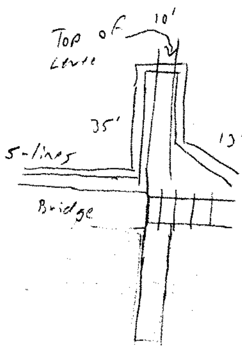


Assume length 500'

LENGTH = 500LF
EXCAVATION

$(.5)(5) = \left(\frac{(2.5SF)(500\,LF)}{27\,CF/CY}\right) = 46$ CY



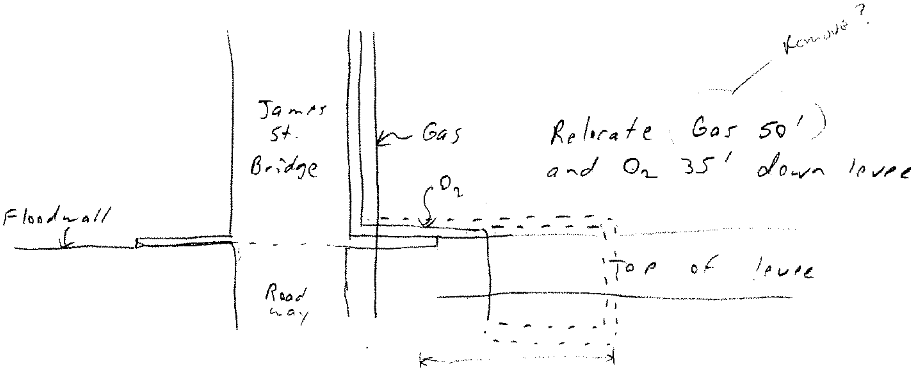


Length of fiber & Conduit Relocation 500'

Quantity of material for Cover ~~870~~ yd³
57

Sta 25+90 & 25+90 O_2 & Gas

Length of pipe relocation = 150' for gas
Length of pipe relocation = 120 for O_2

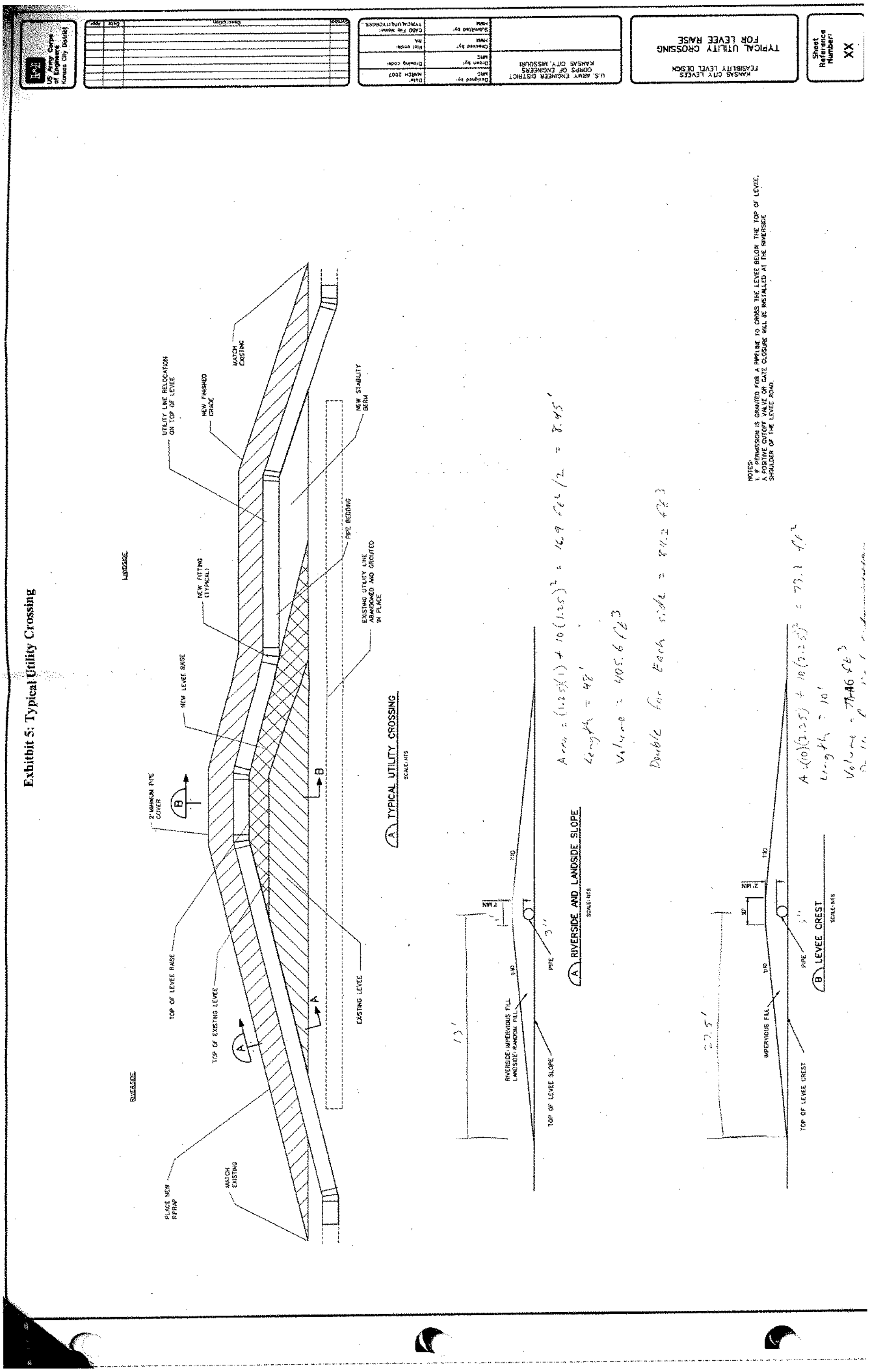


Volume of material for cover 76 cy

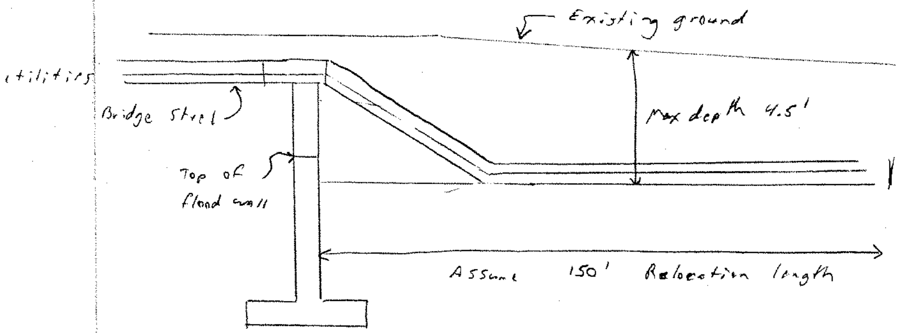
Exc - $\frac{(50)(3)(3)}{27}$ = 17 cy

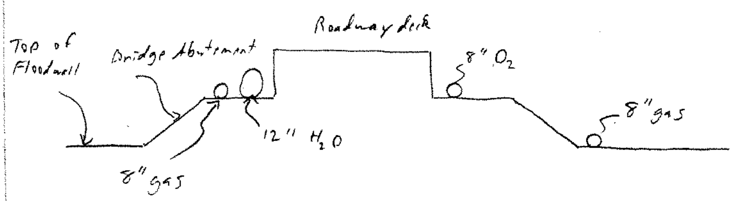
Steel Pile

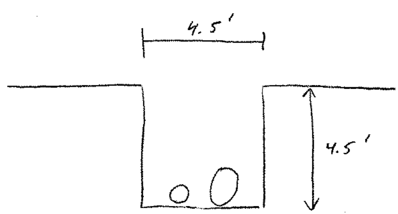
Exhibit 5: Typical Utility Crossing



utilities at 26+10 and 26+10





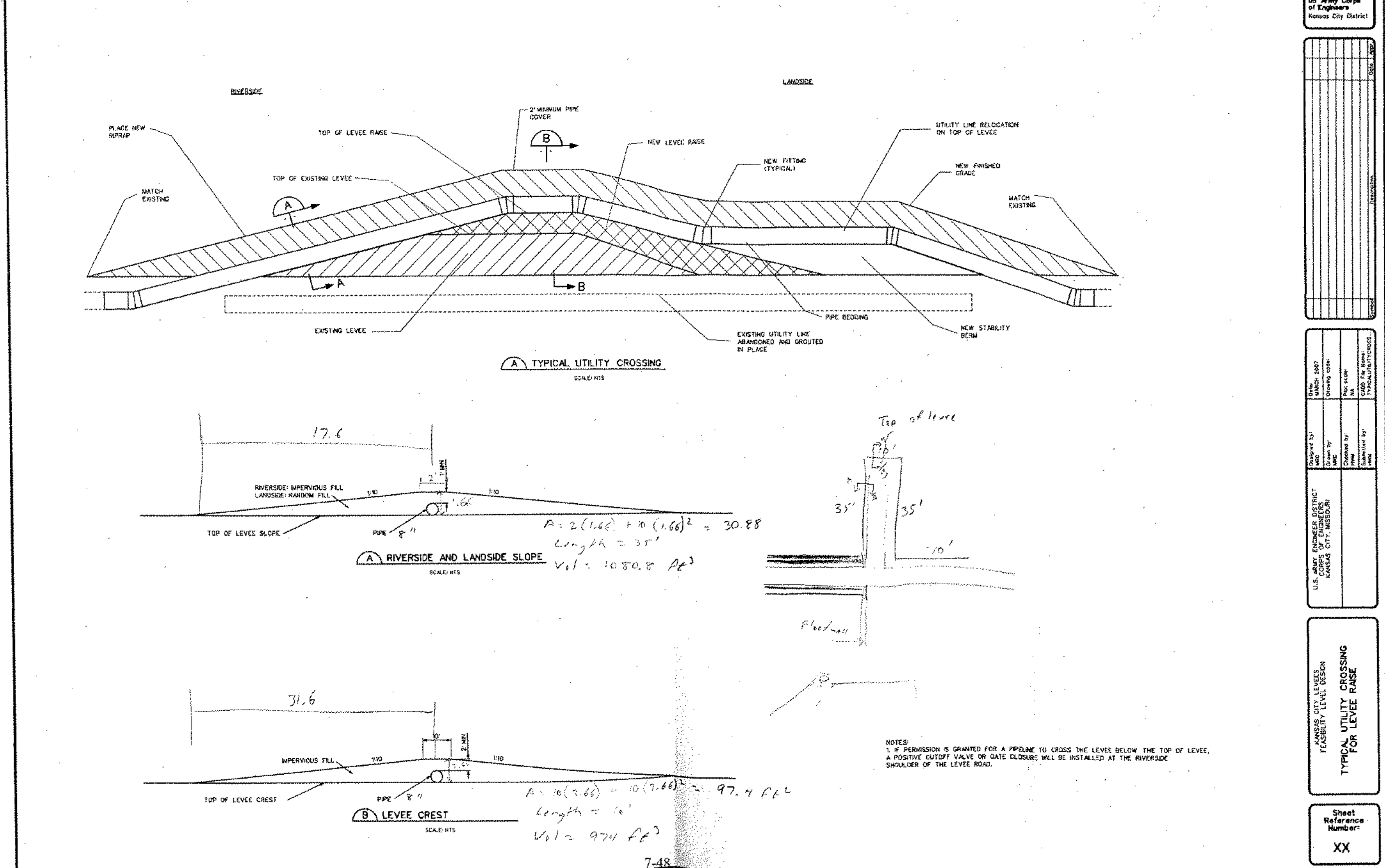


$A = 20.25\ ft^2$
$V = 3037.5\ ft^3$
$(112.5\ yd^3)$

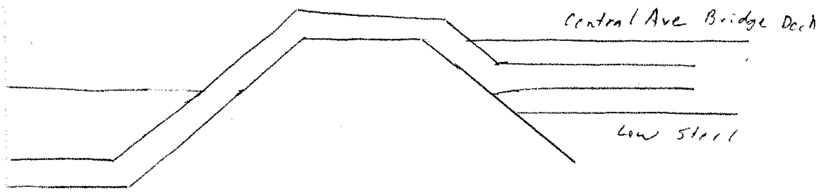
Total length of pipe relocate 150'

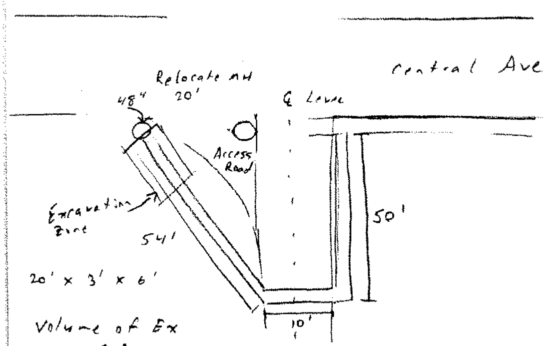
Volume of Excavation 112.5 yd^3

Exhibit 5: Typical Utility Crossing



STA 57+10

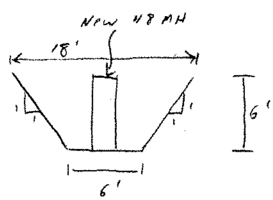




What is exc
How much demo
How much Backfill

20' x 3' x 6'

Volume of Ex
360 ft³
14 cy



Total length of pipe relocation 110'

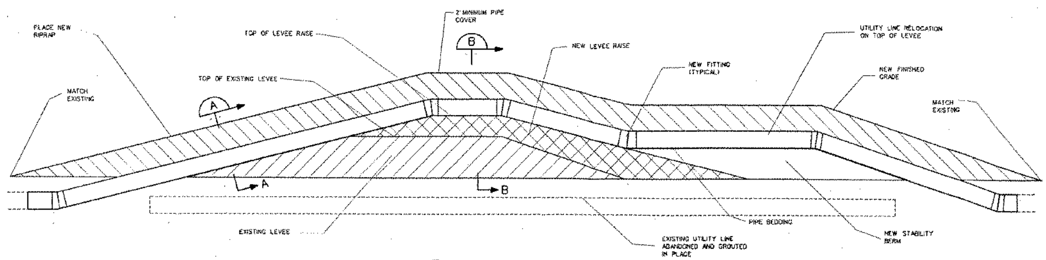
Replace 1 - 48" diam MH @ 6' depth

Volume of Cover 174 cy

Volume of Excavation 54 cy

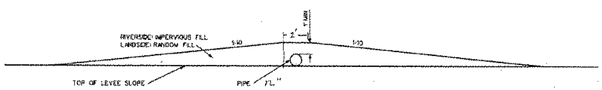
Volume 848 ft³
31 cy

Exhitbit 5: Typical Utility Crossing



A TYPICAL UTILITY CROSSING

SCALE: NTS



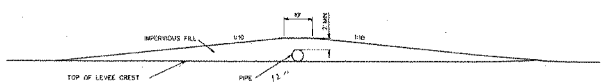
A RIVERSIDE AND LANDSIDE SLOPE

SCALE: NTS

Riverside
Length = 50'
Area = (2)(2) + ½(2)(20) = 22
Volume = 1100 ft³

Landside
Length = 54'
Area = (2)(2) + 2(20) = 44
Volume = 2376 ft³

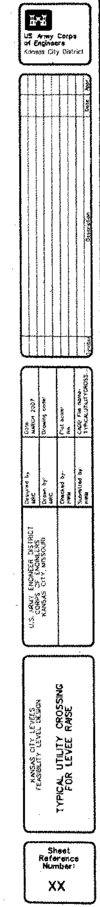
Top
Length = 10'
Area = 3(10) + 30(3) = 120'
Volume = 1200 ft³



B LEVEE CREST

SCALE: NTS

NOTES:
1. IF PERMISSION IS GRANTED FOR A PIPELINE TO CROSS THE LEVEE BELOW THE TOP OF LEVEE, A POSITIVE CUTOFF VALVE OR GATE CLOSURE WILL BE INSTALLED AT THE RIVERSIDE SHOULDER OF THE LEVEE ROAD.

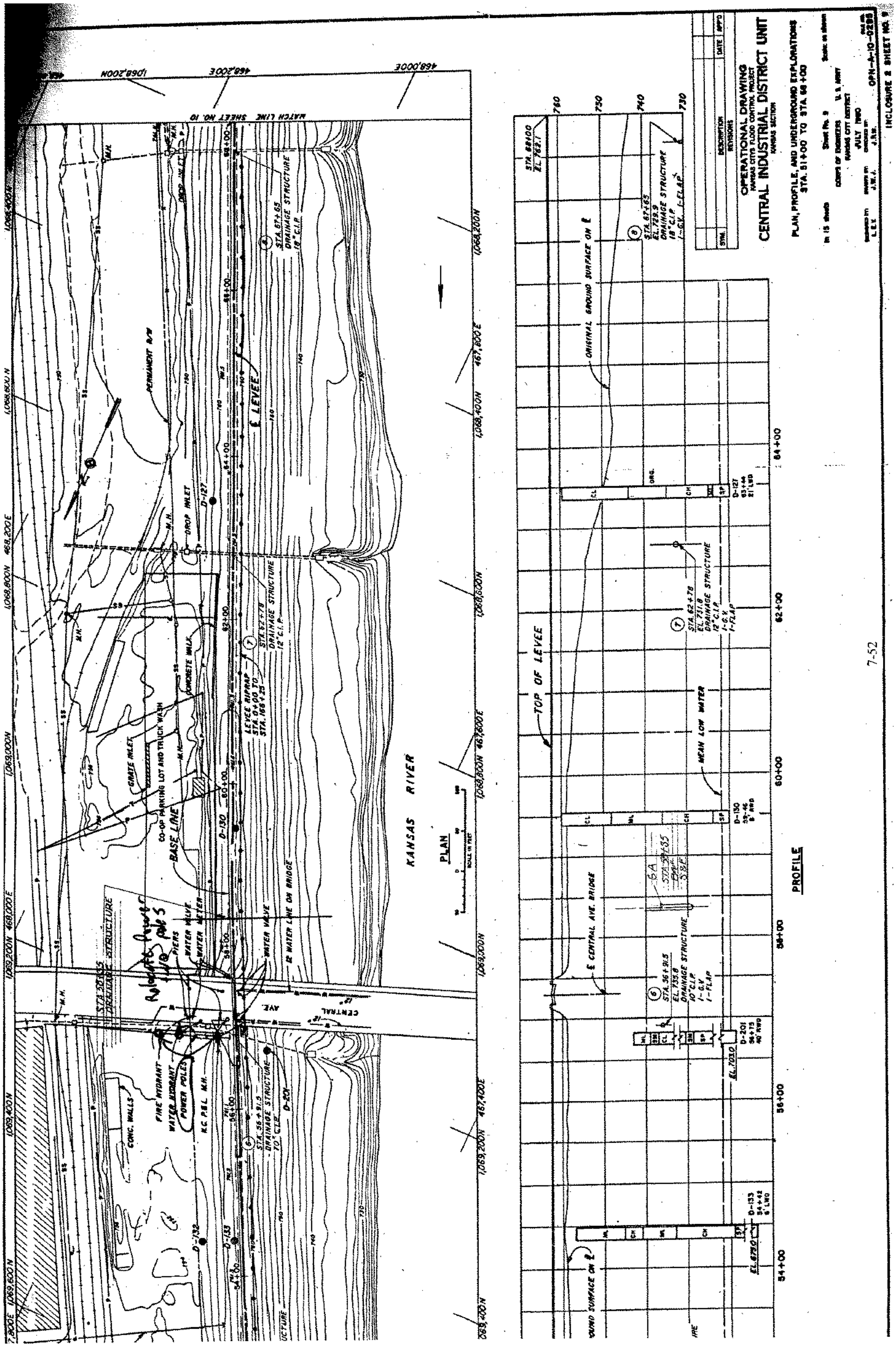


Sta 57+10	Power	

Power comes from wood pole & attaches to overhead steel on the bridge.

Assume relocation of two poles & approx 300' of line

CAMPAD

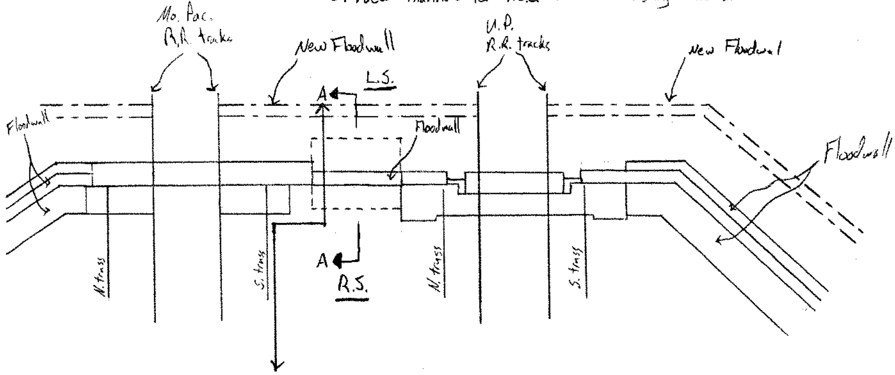

OPERATIONAL DRAWING
CENTRAL INDUSTRIAL DISTRICT UNIT
KANSAS SECTION
PLAN, PROFILE, AND UNDERGROUND EXPLORATIONS
STA. 51+00 TO STA 68+00
INCLOSURE 2 SHEET NO. 9
KANSAS RIVER
PLAN
PROFILE
TOP OF LEVEE
BASE LINE
MEAN LOW WATER
ORIGINAL GROUND SURFACE ON ℄
℄ LEVEE
WATCH LINE SHEET NO. 10
CO-OP PARKING LOT AND TRUCK WASH
℄ CENTRAL AVE. BRIDGE
54+00
56+00
58+00
60+00
62+00
64+00

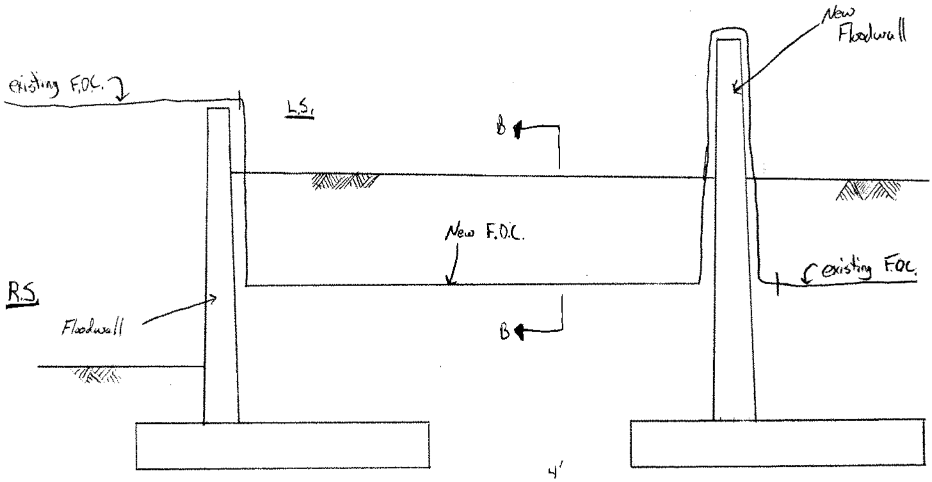
75+50 UL-31 75+60 UL-32

Assumptions

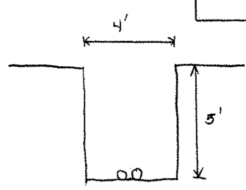
1. Closest tie in is 4000 ft.
2. New manhole for F.O.C. 100' from existing floodwall

FOC





SECTION A
Scale 3/16" = 1'

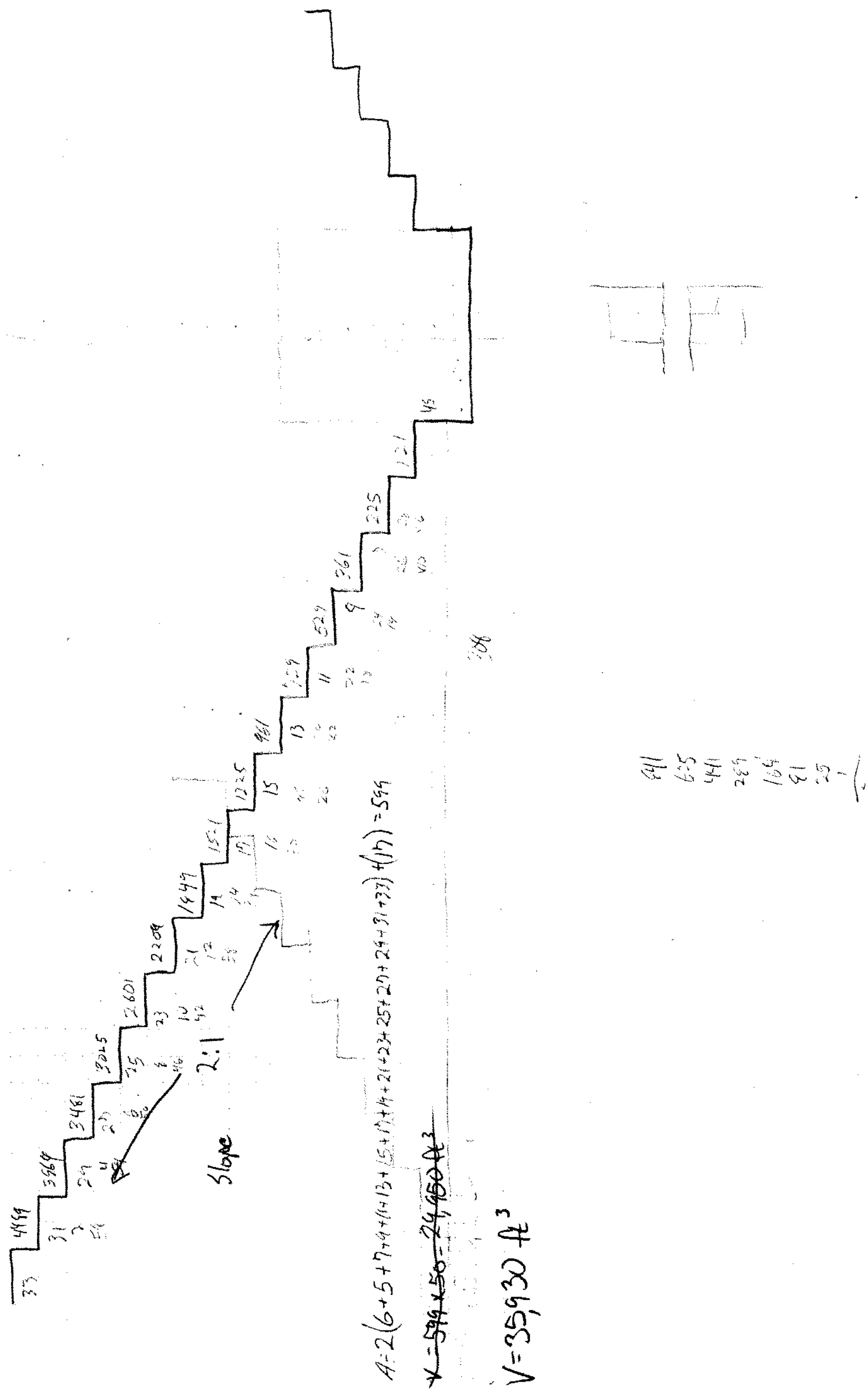


$A = 20\ ft^2$

$V = 20ft^2(100ft)$

$\boxed{V = 2000ft^3}$

Section B

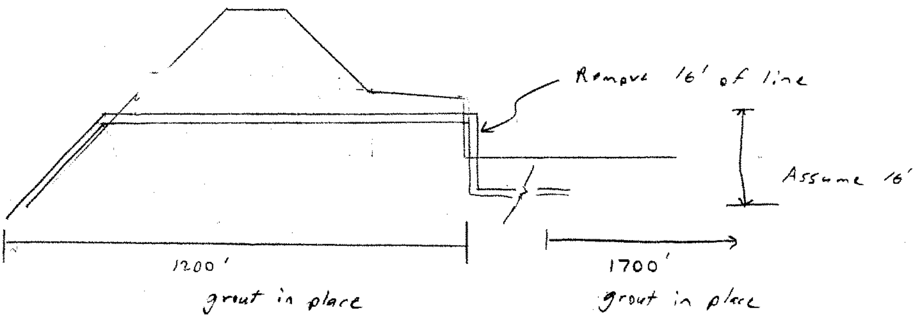

Slope
2:1
V=35,930 ft³

Total Length of new conduit 100'

Total Length of restringing fibers 4000'

Total Excavation 2000 ft^3, ~ 75 CY

Sta 85+20

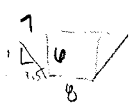


Excavation required minor

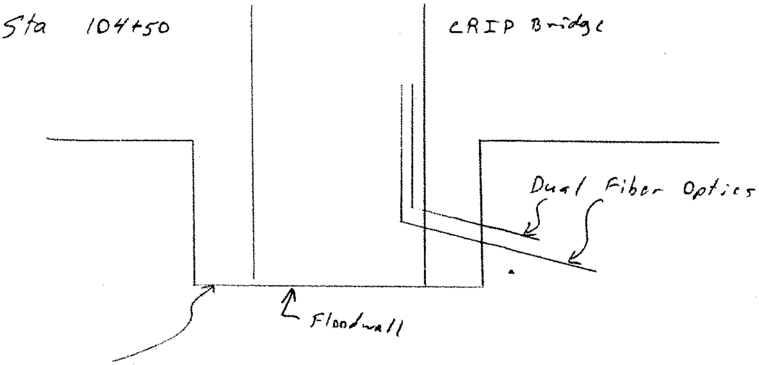
8" Line
Length 2900'

Quantity of grout = 1020 ft^3

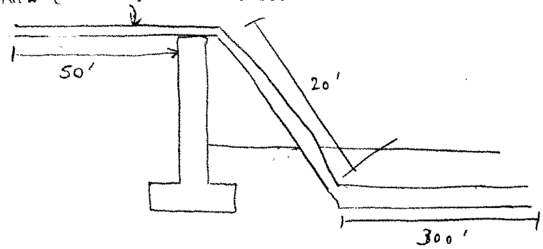
Assume 3 Pits



102 SF x 16
= ~~305~~ 60 cy/Pit
100 cy



1) Build this section of floodwall first

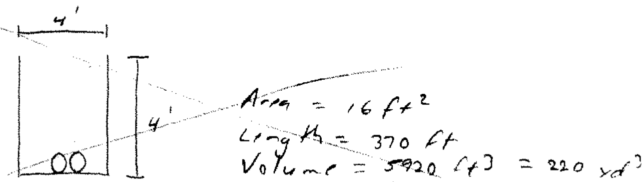


Total length of conduit replacement 50'

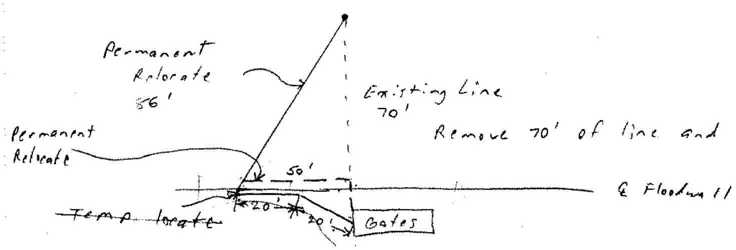
Restring fibers approx length 1000'

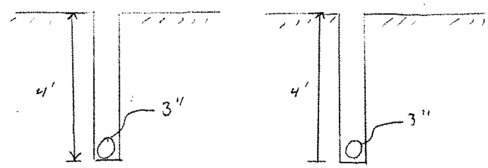
Excavation

Not Needed

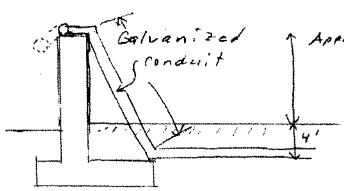


Sta 124 + 60 ~~53~~ Power Relocation





$\frac{(136)(4)(2)}{27} = 40$

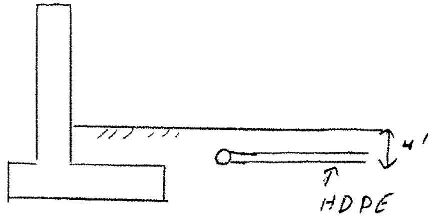


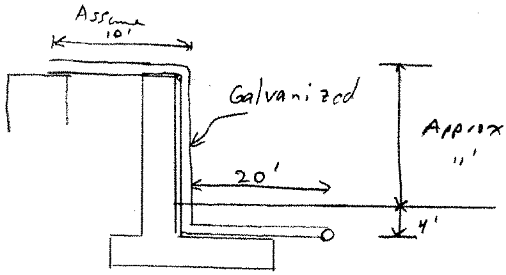
Temp

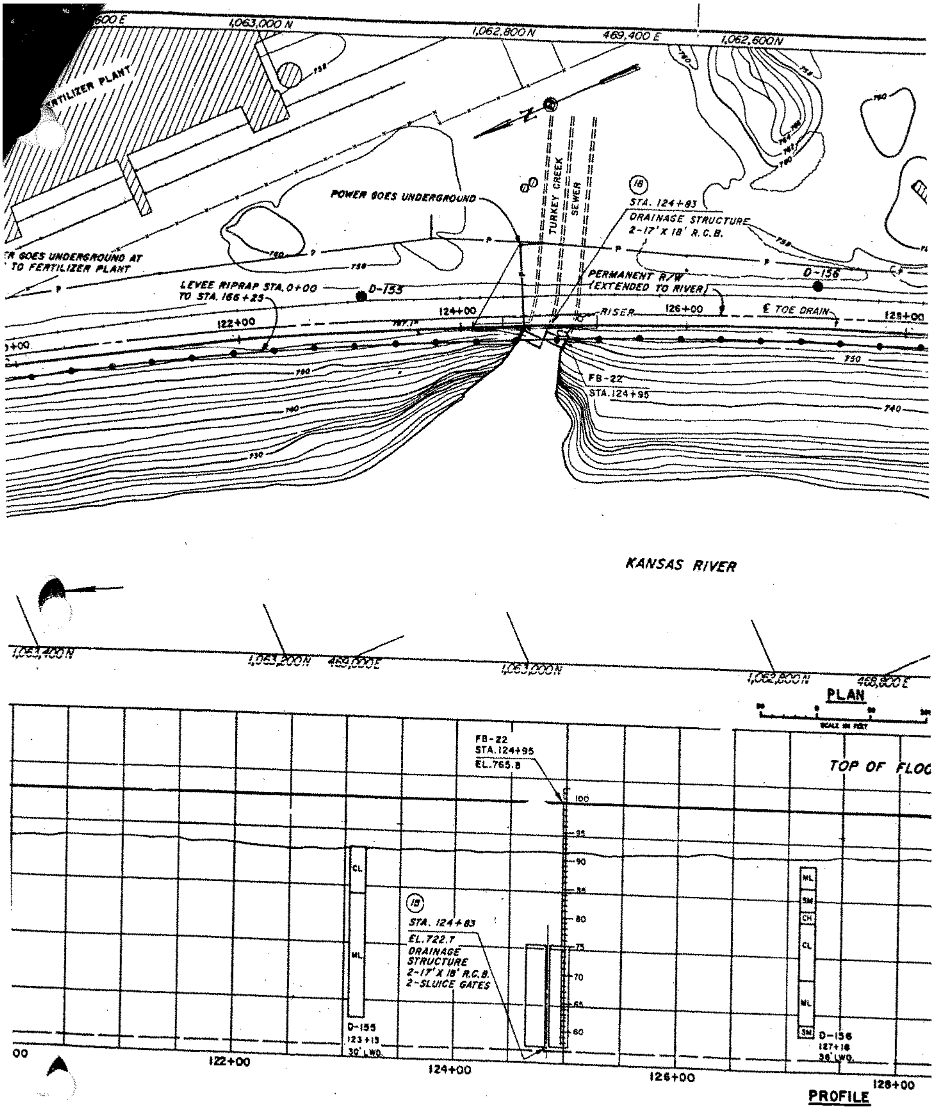
1) Relocate power 86' HDPE to toe of Floodwall
2) Temp locate 60' Galvanized up and over floodwall

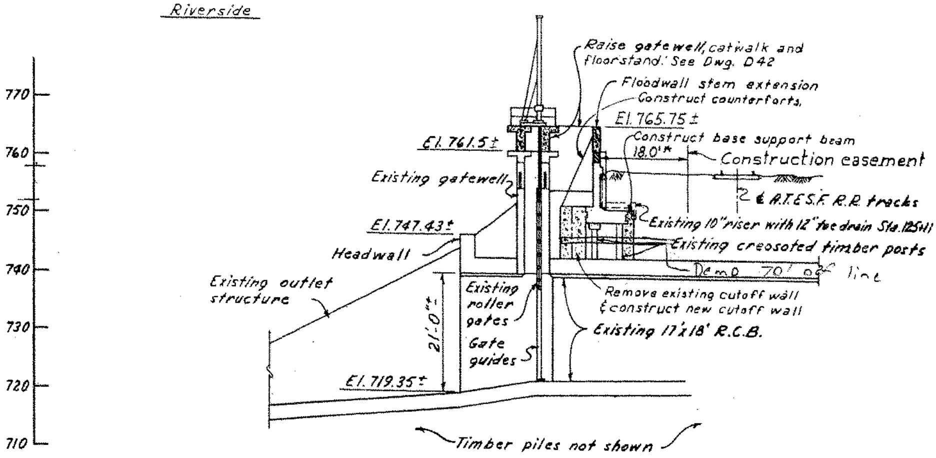
3) Relocate Temp 60' to permanent 95'. Galvanized

Exc - $\frac{(70')(4)(2')}{27} = 21$ cy


4'
HDPE


Assume 10'
Galvanized
Approx 11'
20'
4'


FERTILIZER PLANT
1,063,000 N
1,062,800 N
469,400 E
1,062,600 N
POWER GOES UNDERGROUND
TURKEY CREEK
SEWER
STA. 124+83
DRAINAGE STRUCTURE
2-17' X 18' R.C.B.
LEVEE RIPRAP STA. 0+00
TO STA. 166+25
D-155
D-156
PERMANENT R/W
(EXTENDED TO RIVER)
RISER
TOE DRAIN
122+00
124+00
126+00
128+00
FB-22
STA. 124+95
KANSAS RIVER
1,063,400 N
1,063,200 N
469,000 E
1,063,000 N
1,062,800 N
468,800 E
PLAN
TOP OF FLOO
FB-22
STA. 124+95
EL. 765.8
STA. 124+83
EL. 722.7
DRAINAGE
STRUCTURE
2-17' X 18' R.C.B.
2-SLUICE GATES
D-155
123+13
30' LWO.
D-156
127+18
36' LWO.
PROFILE



SECTION THRU ℄ TURKEY CREEK SEWER
STATION 124+83±

NOTE:
For levee section details, see Dwgs. B1 thru B5

RECORD DRAWING

EXHIBIT A-7.6

CID-KS Utility and Drainage Crossings: Inventory and Action for N500+3 Raise

CID-KS Utility and Drainage Crossings: Inventory and Action for N500+3 Raise
Created by: Cassidy Garden   Peer Reviewed by: Hank Mildenberger

Date: 10-June-08

		CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES										
Map Ref. #	Station (ft.)	Conduit Size	Conduit Function	Flow Direction	Flow Type	Conduit Composition	Conduit Length Below Flood Protection (ft.) Existing Conditions	Control Structure Type	Elev of Invert	Owner/ Compensable Interest	Comments	Action
1	(83+22LE CID MO)	42"	Ohio Ave. Pump Plant Abandoned Outfall	Land to Riv	SEE PUMP STATION EVAL	RCP	130	NA	*		Abandoned 42" Conc. Conduit Filled with Sand. Conc plug on riverside just before gatewell.	No Action
2	-5+85 (83+52 LE CID MO)	42"	Ohio Ave. Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL	RCP	100	Sluice Gate and Flap Gate	730.5	KCK		See Pump Station Evaluation
3	-1+77 (87+60 LE CID MO)	66"	Treated WW Plant Effluent	Land to Riv	Gravity	RCP	60	Sluice Gate	745.69	KCK		See Pump Station Evaluation
4	10+81	*	Drainage Structure	Land to Riv	Gravity	RCP	*	Sluice Gate	746	KCK	Treatment Plant Diversion.	See Structural Evaluation
5	18+10	NA	Fiber Optic	NA	NA	*	*	NA	*	Sprint	Crosses Railroad Bridge.	Relocate up and over the levee. Assume closest tie-in point is 500 feet.
6	Approx 18+20	*	Fiber Optic	NA	NA	*	*	NA	*		Cross Railroad Bridge, see photos CIDKS 08 & 09	Relocate up and over the levee. Assume closest tie-in point is 500 feet.
7	18+75	NA	Fiber Optic	NA	NA	*	*	NA	*	Level 3 Comm	Crosses Railroad Bridge.	Relocate up and over the levee. Assume closest tie-in point is 500 feet.
8	18+75	NA	Fiber Optic	NA	NA	*	*	NA	*	Williams Comm	Crosses Railroad Bridge.	Relocate up and over the levee. Assume closest tie-in point is 500 feet.
9	19+85	3"	Fiber Optic and Railroad Communication	NA	NA	Concrete Tube	*	NA	753		Underground 3" comm line is either active or abandoned in place since previous research indicates a comm line on the Lewis and Clark Viaduct.	This line will have to be relocated up and over based on the levee raise in this area.
10	19+90	66"	Sanitary Sewer	Riv to Land	Gravity: Forcemain on one side of river, gravity sewer on other side	RCP	*	Sluice Gate	748.5	KCK	Sanitary sewer consists of 2-36" CIP under KS River from the Armourdale Forcemain Interceptor and the Fairfax Forcemain Interceptor to a CID-KS Riverside Gatewell.	See Structural Evaluation
	Approx 22+50	2 lines	Overhead Power	NA	NA	n	NA	NA	NA			No Action, these are large overhead power poles that cross the Kansas River. No concerns with clearance.

		CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES										
Map Ref. #	Station (ft.)	Conduit Size	Conduit Function	Flow Direction	Flow Type	Conduit Composition	Conduit Length Below Flood Protection (ft.) Existing Conditions	Control Structure Type	Elev of Invert	Owner/ Compensable Interest	Comments	Action
12	25+90	12"	Gas	NA	Pressure	*	*	*	751	*	Abandoned Main	No Action
13	Approx 25+90	8"	Oxygen	*	Pressure	Steel	*	*	760	Praxair	Located on Bridge. Utility goes up and over floodwall and penetrates embankment on landside of wall. The utility is assumed to be top of wall which is 760.	This line will have to be relocated up and over based on the levee raise in this area. Approximate length of relocation 120 feet.
14	Approx 25+90	8"	Gas	NA	Pressure	Steel	*	*	760		Located on Bridge. Utility goes up and over floodwall and penetrates embankment on landside of wall. The utility is assumed to be top of wall which is 756.	This line will have to be relocated up and over based on the levee raise in this area. Approximate length of relocation 150 feet.
15	26+10	8"	Gas	NA	Pressure	Steel	*	*	760.8		Located on Bridge. Utility goes up and over floodwall and penetrates embankment on landside of wall. The utility is assumed to be top of wall which is 760.8.	Relocate line to accommodate raise. Approximate length of relocation 150 feet.
16	26+10	12"	Water	NA	Pressure	Steel	*	2-12" water valves & 2 - 1 1/2" Drain Valves	760.8		Located on Bridge. Utility goes up and over floodwall and penetrates embankment on landside of wall. The utility is assumed to be top of wall which is 760.8.	Relocate line to accommodate raise. Approximate length of relocation 150 feet.
17	37+07	18"	Mistletoe Yards Pump Plant outfall	Land to Riv	SEE PUMP STATION EVAL	CIP	*	Sluice Gate and Flap Gate	733.5			See Pump Station Evaluation
18	50+98	10"	Storm Sewer	Land to Riv	Gravity	CIP	*	Gate Valve and Flap Gate	734.43			See Structural Evaluation
	52+07	10"	Storm Sewer	Land to Riv	Gravity	CIP	*	Gate Valve and Flap Gate	733.73			See Structural Evaluation

CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES

Map Ref. #	Station (ft.)	Conduit Size	Conduit Function	Flow Direction	Flow Type	Conduit Composition	Conduit Length Below Flood Protection (ft.) Existing Conditions	Control Structure Type	Elev of Invert	Owner/ Compensable Interest	Comments	Action
20	56+92	10"	Storm Sewer	Land to Riv	Gravity	CIP	*	Gate Valve and Flap Gate	735.8			See Structural Evaluation
21	57+10	12"	Water	NA	Pressure	*	*	*	NA		Located on Central Avenue Bridge with valves and meter.	Relocate line to accommodate raise. Approximate length of relocation 110' feet. Replace 48" MH @ 6' depth
22	57+50	12"	Water	NA	Pressure	*	*	*	NA		No evidence of being in existence	No Action
23	57+10	NA	Power	NA	NA	NA	NA	NA	NA			Power comes from wood pole and attaches to overhead steel on the bridge. Some modification will need to be made to avoid this during construction. Assume relocation of 2 poles and approximate 300' of line.
24	58+35	84"	KCK Flood Pump Station #16 (New Central Avenue Pump Plant) outfall	Land to Riv	SEE PUMP STATION EVAL	RCP	*	Sluice Gate, 3-Gate Valves and Flap Gate	729	KCK		See Pump Station Evaluation
25	Approx 59+03	12"	*	*	*	CIP	*	*	738		Source of Information is Record Drawings. Likely has been removed.	No Action
26	62+78	12"	Storm Sewer	Land to Riv	Gravity	CIP	*	Gate Valve and Flap Gate	731.8		This drainage structure has a drop	See Structural Evaluation

		CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES										
Map Ref. #	**Station (ft.)**	**Conduit Size**	**Conduit Function**	**Flow Direction**	**Flow Type**	**Conduit Composition**	**Conduit Length Below Flood Protection (ft.) Existing Conditions**	**Control Structure Type**	**Elev of Invert**	**Owner/ Compensable Interest**	**Comments**	**Action**
27	67+65	18"	Storm Sewer	Land to Riv	Gravity	CIP	*	Gate Valve and Flap Gate	729.9			Raise Existing Manhole 4 feet. See Structural Evaluation for gatewell.
28	71+70	18"	Storm Sewer	Land to Riv	Gravity	CIP	*	Gate Valve and Flap Gate	729.4			See Structural Evaluation
29	74+21	24"	Stock Yards #3 Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL	CIP	*	Sluice Gate and Flap Gate	729.4			See Pump Station Evaluation
30	Approx 74+60	15"	*	*	*	CIP	*	*	*		Record Drawings Indicate this has been removed.	No Action
31	75+50	NA	Fiber Optic	NA	NA	*	NA	NA	*	IXC Comm	Located on Missouri Pacific Bridge	Relocate up and over floodwall. Assume closest tie-in point is 4000 feet.
32	75+60	NA	Fiber Optic	NA	NA	*	NA	NA	*	MCI	Located on Missouri Pacific Bridge	Relocate up and over floodwall. Assume closest tie-in point is 4000 feet.
33	77+80	42"	Storm Sewer	Land to Riv	Gravity	RCP	*	Sluice Gate and Flap Gate	725.5			See Structural Evaluation
34	80+90	60"	Gateway 2000 Pump Plant Outfall	Land to Riv	SEE PUMP STATION EVAL	RCP	*	Sluice Gate and Flap Gate	NA	Gateway 2000		See Pump Station Evaluation

CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES

Map Ref. #	Station (ft.)	Conduit Size	Conduit Function	Flow Direction	Flow Type	Conduit Composition	Conduit Length Below Flood Protection (ft.) Existing Conditions	Control Structure Type	Elev of Invert	Owner/ Compensable Interest	Comments	Action
35	84+90	18"	South Stockyards Conduit 1:	Land to Riv	SEE PUMP STATION EVAL	CIP	*	Sluice Gate and Flap Gate	725.5		The pipe drains the central portion of the south stockyards area immediately adjacent to the levee.	See Pump Station Evaluation
36	85+20	8"	Water	NA	Pressure	*	*	*	*		Water line up wall	Cut line on River Side toe of levee. Remove 16' of pipe at retaining wall. Grout approx 2900' of length.
37	88+19	24"	South Stockyards Conduit 2:	Land to Riv	Gravity	CIP	*	Sluice Gate and Flap Gate	728		The pipe drains the south-central portion of the south stockyards area immediately adjacent to the levee.	No Civil Action. See Structural Evaluation
38	94+32	18"	South Stockyards Conduit 3:	Land to Riv	Gravity	CIP	*	Sluice Gate and Flap Gate	732			No Civil Action. See Structural Evaluation
39	98+05	24"	Stock Yards #1 Outfall	Land to Riv	Gravity	CIP	*	Sluice Gate and Flap Gate	726.5			No Civil Action. See Structural Evaluation
40	102+52	12"	American Royal Drive: The flow drains off of the road along the American Royal area.	Land to Riv	SEE PUMP STATION EVAL	CIP	*	Sluice Gate and Flap Gate	730.8			No Civil Action. See Structural Evaluation
41	104+50	NA	Fiber Optic	NA	NA	*	*	NA	*	Williams Comm		Relocate up and over the floodwall. Assume closest tie-in point is 300 feet.
42	106+49	6'x6'	Kemper Arena Pump Plant outfall	Land to Riv	Gravity	RCB	*	Sluice Gate and Flap Gate	726.7	KCMO		No Civil Action. See Structural Evaluation

		CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES										
Map Ref. #	Station (ft.)	Conduit Size	Conduit Function	Flow Direction	Flow Type	Conduit Composition	Conduit Length Below Flood Protection (ft.) Existing Conditions	Control Structure Type	Elev of Invert	Owner/ Compensable Interest	Comments	Action
43	Approx 124+60	*	Power	*	*	*	*	*	NA			Power for Turkey Creek Sewer Gates. Power currently goes under floowall. Relocate power to go over floodwall. Assume 150' length of line relocate and new power pole (wood).
44	124+83	Double 17' x 18'	Turkey Creek sewer outlet	Land to Riv	Gravity	RCB	*	Sluice Gate (LW) and Flap Gates (RW)	722.7	KCMO		See Structural Evaluation
45	Approx 131+30	4-lines	Overhead Power	*	*	*	NA	*	NA			No Action. these are large overhead power poles that cross the Kausas River. No concerns with clearance.
46	138+29	36"	Railroad Yards Conduit 1: The flow is from the railroad tracks area immediately adjacent to the levee.	Land to Riv	Gravity	CIP	*	Sluice Gate and Flap Gate	738.5			See Structural Evaluation
47	152+28	24"	Railroad Yards Conduit 2: The flow is from the railroad tracks area immediately adjacent to the levee.	Land to Riv	Gravity	CIP	*	Sluice Gate and Flap Gate	745.4			See Structural Evaluation
48	159+70	42"	Railroad Yards Conduit 3: The flow is from the railroad tracks area immediately adjacent to the levee.	Land to Riv	Gravity	CIP	*	Sluice Gate and Flap Gate	736.1			See Structural Evaluation
49	Approx 165+72	2"	Unknown	*	*	*	*	*	*		According to the Operational Drawings, this pipe has been plugged.	No Action
50	Approx 166+08	4"	Unknown	*	*	*	*	*	*		According to the Operational Drawings, this pipe has been plugged.	No Action

CID-KS Utility and Drainage Crossings: Inventory and Action for N500+3 Raise **Date: 10-June-08**
Created by: Cassidy Garden Peer Reviewed by: Hank Mildenberger

		CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES										
Map Ref. #	**Station (ft.)**	**Conduit Size**	**Conduit Function**	**Flow Direction**	**Flow Type**	**Conduit Composition**	**Conduit Length Below Flood Protection (ft.) Existing Conditions**	**Control Structure Type**	**Elev of Invert**	**Owner/ Compensable Interest**	**Comments**	**Action**
51	167+95	10"	Southern end of levee unit	Land to Riv	Gravity	CIP	*	Sluice Gate (LW)	751 +/-			See Structural Evaluation

CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES

Map Ref. #	Station (ft.)	Reference Sited o&m Manual (1978) and 1979 Operations Manual (Operational Drawing, Appendix I Record Drawings Vol One)	Reference Sited: HNTB	Reference Sited: Site visits and KVDD	Reference Sited: Wyandotte County CAD files (not QC checked by County)	Other (Inspection Check List, Kansas Gas Service)
1	(83+22LE CID MO)	Abandoned 42" Conc. Conduit Filled with Sand. Conc plug on riverside just before gatewell	Not included in HNTB information			
2	-5+85 (83+52 LE CID MO)	Slice and Flap Gate, Elev 730.5 Station, elevation, size, conduit composition, and control type were verified on Operational Drawings. General area and size verified on Record Drawings.	A. The outlet is located at mile 378.43 on the Missouri River. B. As noted in the pump plant information, sanitary flow no longer contributes to the plant and the percent impervious has increased. However, the total flow reaching the plant remains the same in the Design River Condition because the sewer capacity controls.Sluice Gate (LW) and Flap Gate (RW)	42", City Owned PS outles	General area verified on KCK Sanitary & Storm Mapping.	Station, owner, control type and size were verified on Inspection Check List.
3	-1+77 (87+60 LE CID MO)	Station, elevation, size, conduit composition, and control type were verified on Operational Drawings.	Treatment Plant Outfall	KCK Owner	General area verified on KCK Sanitary & Storm Mapping.	Station, owner, control type and size were verified on Inspection Check List.
4	10+81	Station, elevation, and control type were verified on Operational Drawings. Operational Drawings state "Drainage Structure."	Treatment Plant Diversion	KCK Owner	General area verified on KCK Sanitary & Storm Mapping.	Station, owner, and control type were verified on Inspection Check List.
5	18+10					
6	Approx 18+20					
7	18+75					
8	18+75					
9	19+85	O&Ms show a telephone cable 3" at about the same elevation as the 66" Sanitary Sewer line. According to HNTB, the line is on the Lewis and Clark Viaduct Station, size, and conduit composition were verified on Operational Drawings.	on Lewis and Clark Viaduct			Schlup, Becker & Brennan Consulting Eng: KS River Crossing Sewage Force Main Drawings - 1965 indicates asbestos cement pipe.
10	19+90	Station, size, conduit composition, and control type were verified on Operational Drawings. Operational Drawings also indicate this is a force main and invert elevation 748.5.	Sanitary sewer siphon consists of 2-36" CIP on R.S.		KCK Sanitary & Storm Mapping indicate 2-36" CIPs in this area. FIELD VERIFY	Schlup, Becker & Brennan Consulting Eng: KS River Crossing Sewage Force Main Drawings - 1965 Station, owner, and size were verified on Inspection Check List. Inspection Check List indicate 3-sluice gates which would indicates the possibility of a depressed sewer.
	Approx 22+50					

CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES

Map Ref. #	Station (ft.)	Reference Sited o&m Manual (1978) and 1979 Operations Manual (Operational Drawing, Appendix I Record Drawings Vol One)	Reference Sited: HNTB	Reference Sited: Site visits and KVDD	Reference Sited: Wyandotte County CAD files (not QC checked by County)	Other (Inspection Check List, Kansas Gas Service)
12	25+90					
13	Approx 25+90					
14	Approx 25+90					
15	26+10	Line on Bridge Location and size were verified on Operational Drawings.				KGS shows crossing at bridge, size verified.
16	26+10	2 1.5" drain lines located near 2 12" water lines Location and size were verified on Operation Drawings. Operation Drawings show 2- 1.5" drain valves and 2-12" drain valves located on bridge.				
17	37+07	Location, elevation, size, conduit compostion, and control type were verified on Operational Drawings. Approximate location, size, conduit composition, and approximate elevation have been verified on the Record Drawings.	A. The pipe serves as the outlet for a pump plant which drains the area immediately adjacent to the pump station. B. As noted in the pump plant information, the sanitary flow has been diverted to the treatment plant. No other significant changes have occurred.		General area verified on KCK Sanitary & Storm Mapping.	Control type and size were verified on Inspection Check List. Inspection Check List indicates a stationing of 37+06, a 1' difference. Need to verify there are not 2 utilities in the area.
18	50+98	Location, elevation, size, conduit composition, and control type were verified on Operational Drawings. Approximate location, size, and conduit composition have been verified on the Record Drawings. Record Drawings indicate this pipe is at elevation 736.6' which is more than 2 feet of difference.	A. There is an inlet for this pipe just upstream of the levee. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. There are no indications that significant changes have taken place in the surrounding area. Flow is coming from the property surrounding the industrial buildings in the immediate area.		General area verified on KCK Sanitary & Storm Mapping.	Station, control type and size were verified on Inspection Check List.
19	52+07	Location, elevation, size, conduit composition, and control type were verified on Operational Drawings. Approximate location and conduit composition have been verified on the Record Drawings. Record Drawings indicate this pipe is at elevation 742.1' which is more than 8 feet of difference. Record Drawings indicate 8" pipe.	A. There is an inlet for this pipe just upstream of the levee. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. There are no indications that significant changes have taken place in the surrounding area. Flow is contributed by the low area north of the Central Avenue Bridge.		General area verified on KCK Sanitary & Storm Mapping.	Station and control type were verified on Inspection Check List and size indicates 10".

		CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES				
Map Ref. #	Station (ft.)	Reference Sited o&m Manual (1978) and 1979 Operations Manual (Operational Drawing, Appendix I Record Drawings Vol One)	Reference Sited: HNTB	Reference Sited: Site visits and KVDD	Reference Sited: Wyandotte County CAD files (not QC checked by County)	Other (Inspection Check List, Kansas Gas Service)
20	56+92	Location, elevation, size, conduit composition, and control type were verified on Operational Drawings. Approximate location, size, and conduit composition have been verified on the Record Drawings. Record Drawings indicate approximate elevation of 739.3 feet which is about 3.5 foot of difference.	A. The conduit is just on the north side of the Central Avenue Bridge. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. It is likely that this pipe does not receive much runoff presently because of the Central Avenue Pump Plant.		KCK Sanitary & Storm Mapping indicate two outfalls in this area and one stub. FIELD VERIFY	Control type and size were verified on Inspection Check List. Inspection Check Lists indicate stationing of 56+91, a 1' difference.
21	57+10	Operational Drawings indicate 2 water lines, size verified.	on Central Avenue Bridge			
22	57+50					
23	57+10					
24	58+35	Location, size, and control type were verified on Operational Drawings.	A. The conduit is just on the south side of the Central Avenue Bridge. B. The outfall is not on the Operational Drawing from July, 1980 that HNTB had in the original research. The KAW Valley Drainage District provided HNTB with an Operational Drawing set from July, 1980 that has this outlet drawn in at its location. C. As noted in the pump plant information, design data is limited. The length of the outlet pipe is estimated to be 170 feet from the KAW Valley operational drawings. However, it is not thought that the drainage area has changed in the past 15 years.		General area verified on KCK Sanitary & Storm Mapping.	Station and size were verified on Inspection Check List. Inspection check List indicates in addition to Sluice and Flap Gates, 3-Gate Valves.
25	Approx 59+03	Record Drawings indicate a pipe at this location.				
26	62+78	Location, elevation, size, conduit composition, and control type were verified on Operational Drawings. Approximate location, size, conduit composition, and approximate elevation have been verified on the Record Drawings.	A. The conduit drains the stockyards area south of the Central Avenue Bridge, which now contains a concrete plant. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. C. The existance of the concrete plant has increased the percent impervious, although no estimates can be made.		General area verified on KCK Sanitary & Storm Mapping.	Station, control type and size were verified on Inspection Check List.

		CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES				
Map Ref. #	Station (ft.)	Reference Sited o&m Manual (1978) and 1979 Operations Manual (Operational Drawing, Appendix I Record Drawings Vol One)	Reference Sited: HNTB	Reference Sited: Site visits and KVDD	Reference Sited: Wyandotte County CAD files (not QC checked by County)	Other (Inspection Check List, Kansas Gas Service)
27	67+65	Location, elevation, size, conduit composition, and control type were verified on Operational Drawings. Approximate location, size, conduit composition, and approximate elevation have been verified on the Record Drawings.	A. The conduit drains the stockyards area south of the Central Avenue Bridge and north of the Missouri Pacific Railroad Bridge. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. There are no indications that significant changes have taken place in the surrounding area.		General area verified on KCK Sanitary & Storm Mapping.	Control type and size were verified on Inspection Check List. Inspection Check List indicates a stationing of 67+85, a 20' difference. Need to verify there are not 2 utilities in the area.
28	71+70	O&M says gate valve and sluice. Location, elevation, size, and conduit composition were verified on Operational Drawings. Operation Drawings indicate 1-Gate Valve and 1-Sluice Gate. Approximate location, size, and conduit composition have been verified on the Record Drawings. Approximate elevation is 731.4 feet.	A. The conduit drains the stockyards area south of the Central Avenue Bridge and just north of the Missouri Pacific Railroad Bridge. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. There are no indications that significant changes have taken place in the surrounding area.		KCK Sanitary & Storm Mapping indicate 2 outfalls in this area. FIELD VERIFY	Station, control type and size were verified on Inspection Check List.
29	74+21	Location, elevation, size, conduit composition, and control type were verified on Operational Drawings. Approximate location, size, and conduit composition have been verified on the Record Drawings. Approximate elevation is 723.1 feet.	A. The outfall is just north of the Missouri Pacific Railway Bridge. B. As noted in the pump plant information, the sanitary flow no longer goes to the pump plant and the Gateway 2000 Pump Plant cut off some of the drainage area to the Stockyards Pump Plant (formerly called Stockyards #3 Pump Plant).		General area verified on KCK Sanitary & Storm Mapping.	Station, control type and size were verified on Inspection Check List.
30	Approx 74+60	Source of Information is Record Drawings.				
31	75+50					
32	75+60					
33	77+80	Location, size, and control type were verified on Operational Drawings.	I-670 has a stand-alone stormwater drainage system and is conveyed through the floodwall. Conditions have not changed with respect to this drainage contribution and disposal.	KVDD: 2 flap gates	General area verified on KCK Sanitary & Storm Mapping.	Station and size were verified on Inspection Check List. Inspection Check List indicates 2 flap gates.
34	80+90		A. The Gateway 2000 Pump Plant outfall conduit replaced the Stockyards #2 Pump Plant outfall. B. The pump plant was designed to have certain pumping capacities at the given river stages. Therefore, the exact service area information was not obtainable in a reasonable amount of time. It was estimated from drainage area mapping provided by the designer of the plant. The estimated percent impervious shown in the table was determined by visual inspection of 1996 aerial photographs. The contributing area to the conduit should not have changed since the design of the pipe.		General area verified on KCK Sanitary & Storm Mapping.	Station, control type, and size were verified on Inspection Check List.

		CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES				
Map Ref. #	Station (ft.)	Reference Sited o&m Manual (1978) and 1979 Operations Manual (Operational Drawing, Appendix I Record Drawings Vol One)	Reference Sited: HNTB	Reference Sited: Site visits and KVDD	Reference Sited: Wyandotte County CAD files (not QC checked by County)	Other (Inspection Check List, Kansas Gas Service)
35	84+90	Location, elevation, size, conduit compostion, and control type were verified on Operational Drawings. Approximate location, elevation, size and comments have been verified on the Record Drawings.	A. There is an 18" VCP coming into the gatewell on the landside. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. There are no indications that significant changes have taken place in the surrounding area.		General area verified on KCK Sanitary & Storm Mapping.	Station, control type, and size were verified on Inspection Check List.
36	85+20					
37	88+19	Location, elevation, size, conduit compostion, and control type were verified on Operational Drawings. Approximate location, size, conduit composition, and elevation are verified on the Record Drawings.	A. There is a 24" VCP coming into the gatewell on the landside and two drop inlets upstream. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. There are no indications that significant changes have taken place in the surrounding area.		General area verified on KCK Sanitary & Storm Mapping.	Station, control type, and size were verified on Inspection Check List.
38	94+32	Location, elevation, size, conduit compostion, and control type were verified on Operational Drawings. Approximate location, size, conduit composition, and elevation are verified on the Record Drawings.	A. There is an 18" VCP coming into the gatewell on the landside and inlets just upstream. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. There are no indications that significant changes have taken place in the surrounding area.		KCK Sanitary & Storm Mapping indicate 2 outfalls in this area. FIELD VERIFY	Station, control type, and size were verified on Inspection Check List.
39	98+05	Location, elevation, size, conduit compostion, and control type were verified on Operational Drawings. Approximate location, size, conduit composition, and elevation are verified on the Record Drawings.	A. There is a 24" VCP coming into the gatewell on the landside. B. As noted in the pump plant information, part of the contributing flow was cut off by Kemper Arena Pump Plant. However, the pump is likely still inadequate.		General area verified on KCK Sanitary & Storm Mapping.	Station, control type, and size were verified on Inspection Check List.
40	102+52	Location, elevation, size, conduit compostion, and control type were verified on Operational Drawings. Approximate location, size, conduit composition, and elevation are verified on the Record Drawings.	A. There is a 12" VCP coming into the gatewell on the landside from American Royal Drive. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. The drainage area has not changed, as it is a roadway.		General area verified on KCK Sanitary & Storm Mapping.	Station, control type, and size were verified on Inspection Check List.
41	104+50					
42	106+49	Location, elevation, size, conduit compostion, and control type were verified on Operational Drawings.	A. The city of Kansas City, Missouri has very little information on the pump plant. HNTB contacted the designers in charge of runoff disposal from the new Butler Manufacturing building which will be constructed to the north of Kemper Arena. The percent impervious shown is an estimate from the designers. B. As noted in the pump plant information, the Butler Manufacturing building area may drain to the Kemper Arena Pump Plant upon construction.		General area verified on KCK Sanitary & Storm Mapping.	Station, control type, and size were verified on Inspection Check List.

		CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES				
Map Ref. #	Station (ft.)	Reference Sited o&m Manual (1978) and 1979 Operations Manual (Operational Drawing, Appendix I Record Drawings Vol One)	Reference Sited: HNTB	Reference Sited: Site visits and KVDD	Reference Sited: Wyandotte County CAD files (not QC checked by County)	Other (Inspection Check List, Kansas Gas Service)
43	Approx 124+60	Power goes underground at this location				
44	124+83	Location, elevation, size, and conduit compostion were verified on Operational Drawings. Operational Drawings indicate 2 sluice gates.	A. The 25th and Fairmont Pump Station, as well as the Southwest Boulevard Pump Station discharge into this pressure sewer. B. As noted in the pump plant information, sanitary flow has been diverted from contributing pump plants. However, impervious area has increased in the pump plant service areas which has created increased runoff.		General area verified on KCK Sanitary & Storm Mapping.	Station and size were verified on Inspection Check List. Inspection Check List indicates 2 sluice gates.
45	Approx 131+30	Four overhead powerlines				
46	138+29	Location, elevation, size, conduit compostion, and control type were verified on Operational Drawings. Approximate location, size, and conduit composition are verified on the Record Drawings. Record Drawings indicate approximate elevation of 740 feet.	A. The KAW Valley Drainage District indicated that the conduit drains the railroad tracks along the levee. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. There are no indications that significant changes have taken place in the surrounding area.		General area verified on KCK Sanitary & Storm Mapping.	Station, control type, and size were verified on Inspection Check List.
47	152+28	Location, elevation, size, conduit compostion, and control type were verified on Operational Drawings. Approximate location, size, conduit composition, and approximate elevation are verified on the Record Drawings.	A. The KAW Valley Drainage District indicated that the conduit drains the railroad tracks along the levee. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. There are no indications that significant changes have taken place in the surrounding area.		General area verified on KCK Sanitary & Storm Mapping.	Station, control type, and size were verified on Inspection Check List.
48	159+70	Location, elevation, size, conduit compostion, and control type were verified on Operational Drawings. Approximate location, size, conduit composition, and approximate elevation are verified on the Record Drawings.	A. The KAW Valley Drainage District indicated that the conduit drains the railroad tracks along the levee. B. No information was given pertaining to the drainage area of this gravity flow pipe. The missing information can be estimated if deemed necessary. There are no indications that significant changes have taken place in the surrounding area.		General area verified on KCK Sanitary & Storm Mapping.	Station, control type, and size were verified on Inspection Check List.
49	Approx 165+72	Approximate location, and size are verified on the Record Drawings.				
50	Approx 166+08	Approximate location, and size are verified on the Record Drawings.				

		CID-KS UTILITY CROSSINGS, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES				
Map Ref. #	Station (ft.)	Reference Sited o&m Manual (1978) and 1979 Operations Manual (Operational Drawing, Appendix I Record Drawings Vol One)	Reference Sited: HNTB	Reference Sited: Site visits and KVDD	Reference Sited: Wyandotte County CAD files (not QC checked by County)	Other (Inspection Check List, Kansas Gas Service)
51	167+95	Location, elevation, size, conduit compostion, and control type were verified on Operational Drawings.	A. The pipe discharges through the floodwall just on the east side of a stoplog gap. B. The KAW Valley Drainage District does not have any information on this pipe and considers it insignificant.		Not Located on KCK Sanitary & Storm Mapping FIELD VERIFY	Station, control type, and size were verified on Inspection Check List.

		CID-KS CRITICAL ZONE UTILITIES, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES										
Map Ref. #	Station (ft.)	Conduit Size	Conduit Function	Flow Direction	Flow Type	Conduit Composition	Conduit Length Below Flood Protection (ft.) Existing Conditions	Control Structure Type	Elev of Invert	Owner/ Compensable Interest	Comments	Action
	KS, MO state line to approx 5+00	15"	Toe Drain	Toward MO	Gravity	Perforated CMP	140' under toe	None	Varies 750 to 748.3		Flow toward Hannibal Bridge	See Geotechnical Evaluation
	Lewis and Clark Viaduct to James Street Viaduct	12"	Gas	*	Pressure	*	690' under toe	*	751	*	Abandoned Main	Verify during construction that this line have been either removed or plugged. If not either grout in place or remove.
	Approx James Street Viaduct to approx 38+00	NA	Overhead power	landward side	NA	NA	Land side critical zone	NA	NA		Power poles and lines outside Levee RoW	For the area fill remove and replace 8 light poles with single lines.
	approx 50+98 to approx 68+00	36"	Storm Sewer	Parallel to Levee	Gravity	RCP	Land side critical zone	None	Assume 20'		36" Assume RCP. Approximately 200' from centerline of levee	No Action
	Approx 68+00 to Approx 82+00	30"	Storm Sewer	Parallel to Levee	Gravity	CIP	Land side critical zone	None	See notes		At station 68+00 approx 150' from centerline of levee. At station 72+00 approx 200' from centerline of levee. From Approx sta 68+00 to approx sta 81+00 ave depth is 21.5'. From Approx sta 81+00 to 82+00 ave depth is 9'.	Modify Manholes to accommodate fill. Raise precast manholes 5'. Manholes 1, 2c, 5 are 5' diameter. Manhole 4 is 6' diameter. Replace manholes 6 and 7. Manholes 6 and 7 are 6' diameter and 10 and 11 feet deep respectively.
	Approx 82+00 to Approx 94+50	15", 18", 30"	Storm Sewer	Parallel to Levee	Gravity	ESVCP	Land side critical zone	None	varies			Modify Manhole and Sewerline to accommodate fill. Raise Manhole number 9, 10, 11. Manholes 9 and 10 are 5' diameters and manhole 11 is 6' diameter. Raise manholes 6, 5 and 2 feet respectively. Replace 580' of 15" ESVCP, 380' of 18" ESVCP, and 260' of 30" ESVCP with equivalent diameter RCP.
	Approx 98+50 to Approx 103+00	12"	Storm Sewer	Parallel to Levee	Gravity	ESVCP	Land side critical zone	None	14.73			No Action
	Approx 67+65	15"	Lateral Drain	Parallel to Levee	Gravity	ESVCP	Land side critical zone	None	Assume depth of 5'			To be abandoned in place.
	Approx 53+00	NA	Anchor Wire	NA	NA	NA	Land side critical zone	NA	NA			Relocate single wood power pole outside of footprint of levee raise. Approx relocate of 20'

		CID-KS CRITICAL ZONE UTILITIES, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES										
Map Ref. #	Station (ft.)	Conduit Size	Conduit Function	Flow Direction	Flow Type	Conduit Composition	Conduit Length Below Flood Protection (ft.) Existing Conditions	Control Structure Type	Elev of Invert	Owner/ Compensable Interest	Comments	Action
	Approx 53+50	NA	Anchor Wire	NA	NA	NA	Land side critical zone	NA	NA			Relocate single wood power pole outside of footprint of levee raise. Approx relocate of 20'
	Approx 58+00	NA	Billboard	NA	NA	NA	Land side critical zone	NA	NA			Relocate billboard approximately 60'
	Approx 57+25 to 73+80	NA	Power	NA	NA	NA	Land side critical zone	NA	NA			Levee raise will require a relocation of 15 wooden poles and 1800' of line. Relocate to the toe of the levee approx 20' from existing location
	Approx 60+00	NA	Power and Telephone	NA	NA	NA	Land side critical zone	NA	NA			Power and telephone terminate at pump station. No action required.
	Gateway 2000Pump Plant to Field Pump House	NA	Power	*	*	*	*	*	NA			Remove Power poles and lines and do not replace. Approximately 600' minimum of 3 strands and 4 poles.
	Approx 85+20 to approx 101+00	4", 8" & 10"	Water	*	*	*	*	*	*		Runs parallel to levee approx 100' off centerline, landward and services Kansas City Stock Yards.	Grout waterlines. Assume Max depth of 6'. Total length of grout 2900'
	Approx 98+00 to Approx 106+00	4" & 8"	Water	*	*	*	*	*	*		Water lines within critical zone.	Included in the length above
	Approx 120+00 to Approx 133+00	NA	Power	*	*	*	*	*	*		Power lines run parallel to levee approximately 75' off centerline, landward.	Power services Turkey Creek Sto
	Approx 146+00 to unknown		Interior drainage								Interior drainage with drop inlets running parallel to levee approximately 50' off center.	No Action
	Sprint Parking Storm Drain Mod											Remove subdrainage system including curb inlets and area inlets. Replace after area fill.

CID-KS Utility and Drainage Crossings: Inventory and Action for N500+3 Raise
Created by: Cassidy Garden   Peer Reviewed by: Hank Mildenberger

Date: 10-June-08

		CID-KS CRITICAL ZONE UTILITIES, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES				
Map Ref. #	Station (ft.)	Reference Sited o&m Manual (1978) and 1979 Operations Manual (Operational Drawing, Appendix I Record Drawings Vol One)	Reference Sited: HNTB	Reference Sited: Site visits and KVDD	Reference Sited: Wyandotte County CAD files (not QC checked by County)	Other (Inspection Check List, Kansas Gas Service)
	KS, MO state line to approx 5+00	Needs to be inspected	Located under landward toe of levee and has 3 manholes on this line.			
	Lewis and Clark Viaduct to James Street Viaduct	Groute or remove whichever is cheapest.	Gas main runs parallel to levee from Lewis & Clark Viaduct and crosses levee at 25+90. Not sure if it goes through or over levee on bridge Operational Drawings show this line running parallel to the levee and approx 20' off centerline of the levee and crosses at 25+90. Record Drawings indicate an existing gas line and a lowered gas line and size has been verified.			
	Approx James Street Viaduct to approx 38+00	As long as there is enough clearance to construct the poles are Okay				
	approx 50+98 to approx 68+00	Leave It.	Sanitary Sewer line runs parallel to levee from approx 50+98 to approx 71+70. Sanitary line is located approximately 150' to 200' off center landward of levee.			
	Approx 68+00 to Approx 82+00					
	Approx 82+00 to Approx 94+50					
	Approx 98+50 to Approx 103+00					
	Approx 67+65					
	Approx 53+00	Move	Operational Drawings indicate two anchor wires coming from powerlines, anchoring 12' off centerline and 26' off centerline of levee. **FIELD VERIFY**			

CID-KS Utility and Drainage Crossings: Inventory and Action for N500+3 Raise
Created by: Cassidy Garden Peer Reviewed by: Hank Mildenberger

Date: 10-June-08

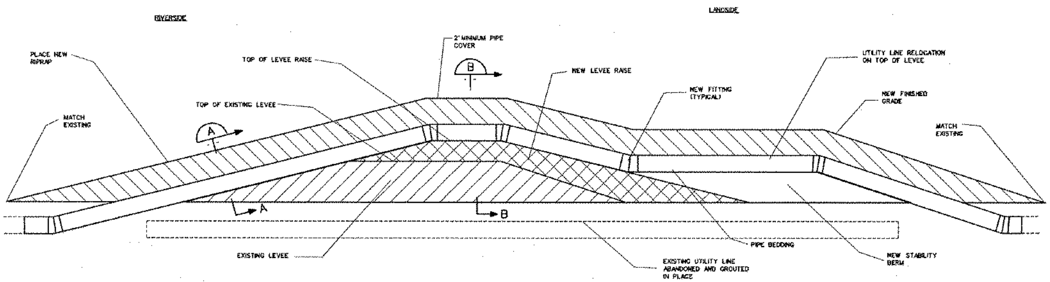
		CID-KS CRITICAL ZONE UTILITIES, GENERAL INFORMATION, AND RECOMMENDED ACTION FOR N500+3 RAISE, BACKGROUND DOCUMENTS AND RESEARCH NOTES				
Map Ref. #	Station (ft.)	Reference Sited o&m Manual (1978) and 1979 Operations Manual (Operational Drawing, Appendix I Record Drawings Vol One)	Reference Sited: HNTB	Reference Sited: Site visits and KVDD	Reference Sited: Wyandotte County CAD files (not QC checked by County)	Other (Inspection Check List, Kansas Gas Service)
	Approx 53+50	Move	Operational Drawings indicate two anchor wires coming from powerlines, anchoring 12' off centerline and 26' off centerline of levee. **FIELD VERIFY**			
	Approx 58+00					
	Approx 57+25 to 73+80	As long as there is enough clearance to construct the poles are Okay	Numerous Power lines run parallel to levee from approximately 53+00 to 73+80 where Well Number 4 is plugged.			
	Approx 60+00	Field Verify, these may go to pump plant	Operational Drawings indicate power and telephone lines 14' off centerline and 42' off centerline of levee. FIELD VERIFY			
	Gateway 2000Pump Plant to Field Pump House	Verify that this line ends at Stock Yards #1 and only services Stock Yards #1 then remove				
	Approx 85+20 to approx 101+00	Determine where water is coming from and find records of abandoment. If abandoned preference is to dig it up.	Water line crosses and then follows parallel to levee approximately 100' of centerline. Ending location is unclear			
	Approx 98+00 to Approx 106+00	Part of system above see notes above.				
	Approx 120+00 to Approx 133+00	Leave as is.				
	Approx 146+00 to unknown					
	Sprint Parking Storm Drain Mod					

CID-KS UTILITY AND DRAINAGE CROSSINGS: INVENTORY LEGEND

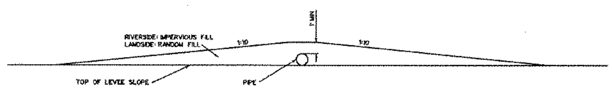
Legend

DS	Drainage Structure	All Drainage Structures that say No Action, mean there is no action recommended for utility outlet relocation.
FW	Floodwall	Structural or pumping analysis may indicate other recommendations and should be referred to for gatewell structural analysis
Land to Riv	Landside to Riverside	Direction of Flow for sanitary sewers was obtained from Wyandotte County Sewer AutoCad files
LEV	Levee	All elevations presented in this spreadsheet are based on NGVD 29 ???????
NA	Not Applicable	See detail "Typical Utility Abandonment - Left In Place" for how to abandon utility
OH	Overhead utility (does not penetrate flood protection)	
PP	Pump Plant	
RCB	Reinforced Concrete Box	
RCP	Reinforced Concrete Pipe	
Riv to Land	Riverside to Landside	
SL	Stop Log Gap	
SP	steel pipe	
*	Information Not Found	
OD	Operational Drawings	
ICL	Inspection Check List	
KGS	Kansas Gas Service	
KCKSS	KCK Sanity & Storm	

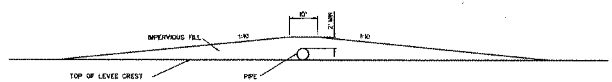
Exhibit A-7.7
Typical Utility Crossing Levee Raise



A TYPICAL UTILITY CROSSING
SCALE: NTS



A RIVERSIDE AND LANDSIDE SLOPE
SCALE: NTS



B LEVEE CREST
SCALE: NTS

NOTES:
1. IF PERMISSION IS GRANTED FOR A PIPELINE TO CROSS THE LEVEE BELOW THE TOP OF LEVEE, A POSITIVE CUTOFF VALVE OR GATE CLOSURE WILL BE INSTALLED AT THE RIVERSIDE SHOULDER OF THE LEVEE ROAD.

US Army Corps of Engineers
Kansas City District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

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Checked by: HNM
Submitted by: HNM
Date:
Drawing code:
Plot scale: NA
CADD File Name: UTILCROSSINGDETAIL.VS

KANSAS CITY LEVEES
FEASIBILITY LEVEL DESIGN
TYPICAL UTILITY CROSSING LEVEE RAISE

Sheet Reference Number: XX

EXHIBIT A-7.8

Storm Drain Modifications Due to Area Fill Requirements

Area Fill 32+50 to 38+00 Area Inlet Modification
Summary

1) Construct 300' New RCP (tie into existing system)

2) Construct 48" diameter Precast drop Inlet

3) Demo existing drop Inlet (48" diam)

4) Excavation for

a) New pipe = 311 yd^3

b) New Inlet = 12 yd^3

c) Demo old Inlet = 7 yd^3

5) Backfill for

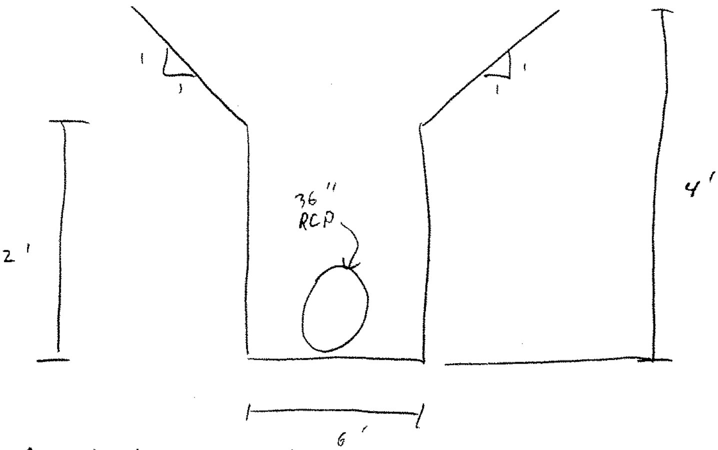
New pipe = 232 yd^3

New Inlet = 7 yd^3

Old Inlet = 12 yd^3

Area Fill 32+50 + 38+00 Inlet Mod

Excavation - pipe



$A = (6)(4) + (2)(2) = 28 ft^2$

$L = 300'$

$V = 8400\ ft^3 = \underline{\underline{311\ yd^3}}$

Back fill - pipe

$A = (6)(4) + (2)(2) - \pi/4 (36/12)^2 = 21\ ft^2$

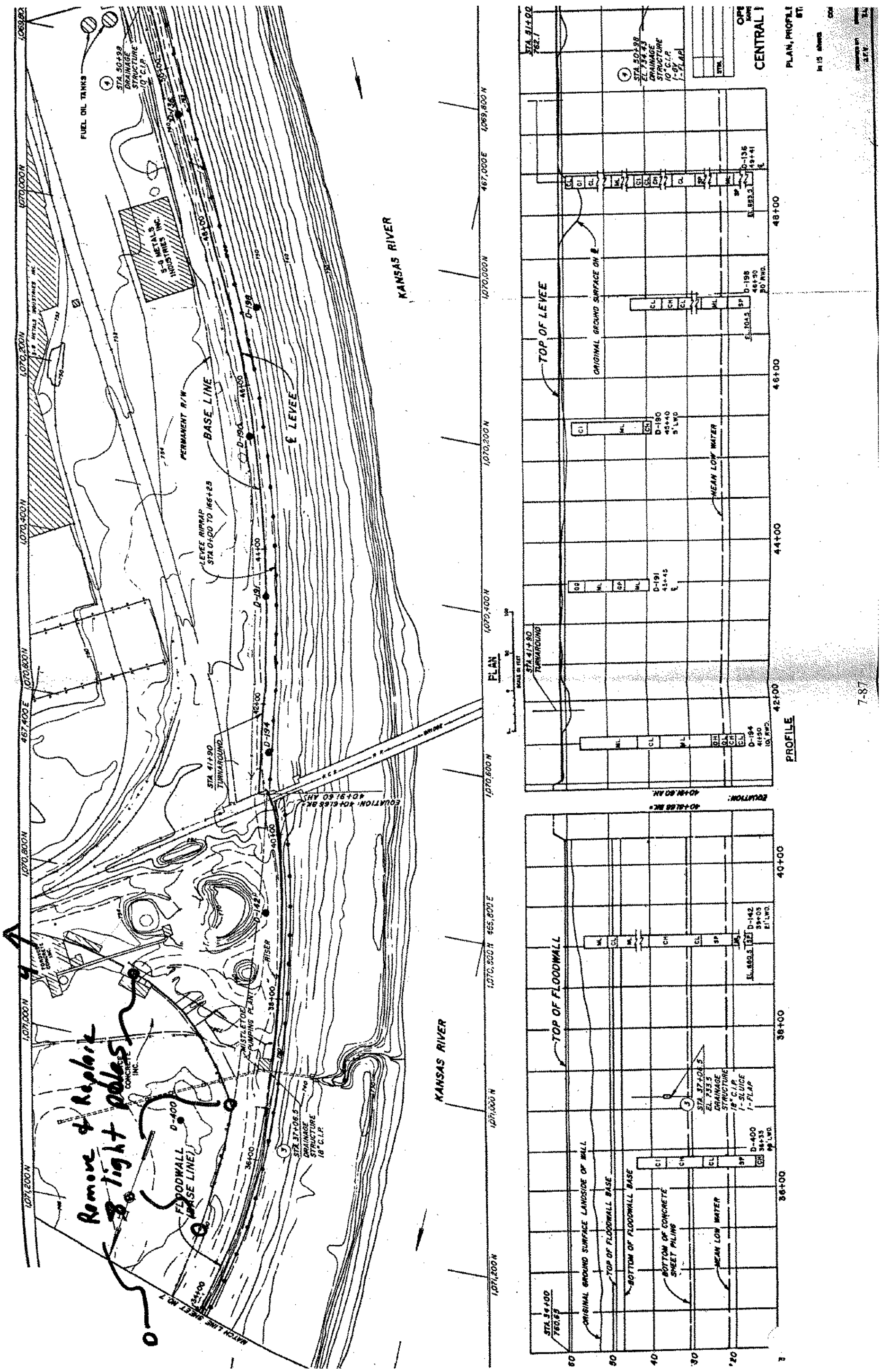
$L = 300'$

$V = 6280\ ft^3 = \underline{\underline{232\ yd^3}}$

Approx James St to Sta 38+00

Remove for area fill.

Replace after paving


KANSAS RIVER
BASE LINE
℄ LEVEE
PERMANENT R/W
FUEL OIL TANKS
S-B METALS INDUSTRIES INC.
STA 41+90 TURNAROUND
FLOODWALL (BASE LINE)
PLAN
PROFILE
TOP OF LEVEE
TOP OF FLOODWALL
ORIGINAL GROUND SURFACE ON ℄
MEAN LOW WATER
BOTTOM OF FLOODWALL BASE
TOP OF FLOODWALL BASE
BOTTOM OF CONCRETE SHEET PILING
CENTRAL
Remove & Replace 3 light poles
7-87

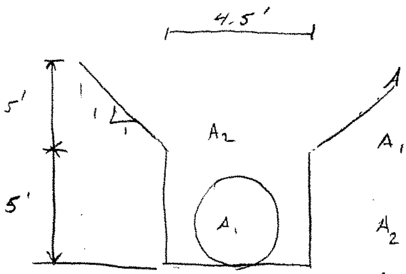
Sprint Parking Subsurface drainage Modification

1) Assume 10 curb inlets to be removed

2) Assume 3 Area inlets to be removed

3) Assume 1000' of concrete pipe to be removed

Excavation Quantity for removal of pipe



Assume All pipe is 36" diameter

$A_1 = \frac{\pi}{4}\left(\frac{36}{12}\right)^2 = 7.07\ ft^2$

$A_2 = (4.5)(10) + (5)(5) = 70\ ft^2$

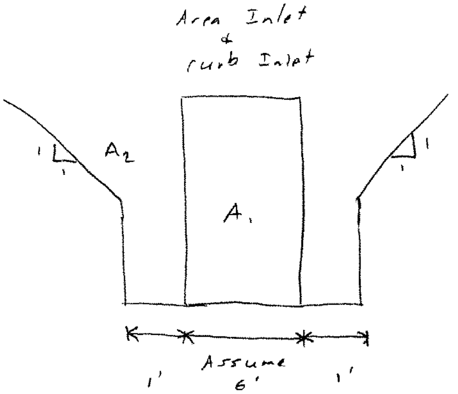
$A_3 = A_2 - A_1 = 70 - 7.07 = 63\ ft^2$

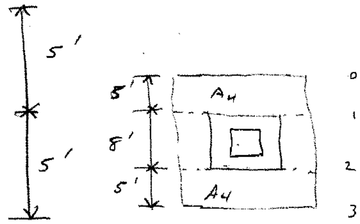
Total Excavation $= \frac{63\ (1000)}{27} = 2340\ yd^3$

Back Fill Quantity for removal of pipe

$\frac{70\ (1000)}{27} = 2600\ yd^3$

Assume all inlets are $6' \times 6' \times 10'$





$$A_1 = (6)(10) = 60 \text{ ft}^2$$

$$A_2 = (8)(10) + (5)(5) = 105 \text{ ft}^2$$

$$A_3 = A_2 - A_1 = 45 \text{ ft}^2$$

$$A_4 = \tfrac{1}{2}(5)(5) = 12.5 \text{ ft}^2$$

Total Excavation Volume

$$\frac{(45)(8)}{27} = 14 \text{ yd}^3$$

$$2\left(\frac{(12.5)(18)}{27}\right) = 17 \text{ yd}^3$$

$$= (31 \text{ yd}^3)(13) = \underline{\underline{403 \text{ yd}^3}}$$

Total Backfill Volume

$$\frac{(105)(8)}{27} = 32 \text{ yd}^3$$

$$2\left(\frac{(12.5)(18)}{27}\right) = 17 \text{ yd}^3$$

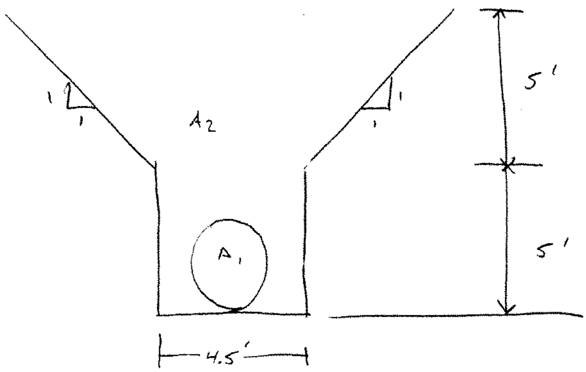
$$= 49 \text{ yd}^3 (13) = \underline{\underline{637 \text{ yd}^3}}$$

1) Assume 7 new curb inlets

2) Assume 1 new area inlet

3) Assume 1075' new 36" diameter concrete pipe

Excavation Quantity for new pipe



$A_1 = \frac{\pi}{4}\left(\frac{36}{12}\right)^2 = 7.07\ ft^2$

$A_2 = (4.5)(10) + (5)(5) = 70\ ft^2$

$A_3 = A_2 - A_1 = 63\ ft^2$

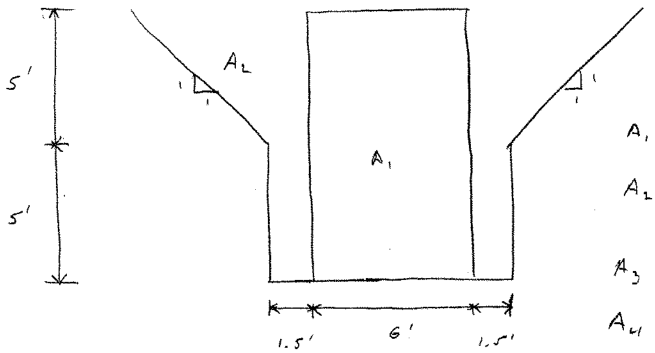
Total Excavation for new pipe

$\frac{70(1075)}{27} = 2800\ yd^3$

Back fill Quantity for new pipe

$\frac{63(1075)}{27} = 2510\ yd^3$

Assume All curb inlets and area inlet is $6' \times 6' \times 10'$

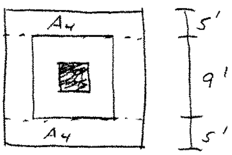


$A_1 = (6)(10) = 60\ ft^2$

$A_2 = (9)(10) + (5)(5)$
$= 115\ ft^2$

$A_3 = A_2 - A_1 = 55\ ft^2$

$A_4 = \frac{1}{2}(5)(5) = 12.5\ ft^2$



Total Excavation for new inlets

$$\frac{(115)(9)}{27} = 40\ yd^3$$

$$2\,\frac{(12.5)(5)}{27} = 5\ yd^3$$

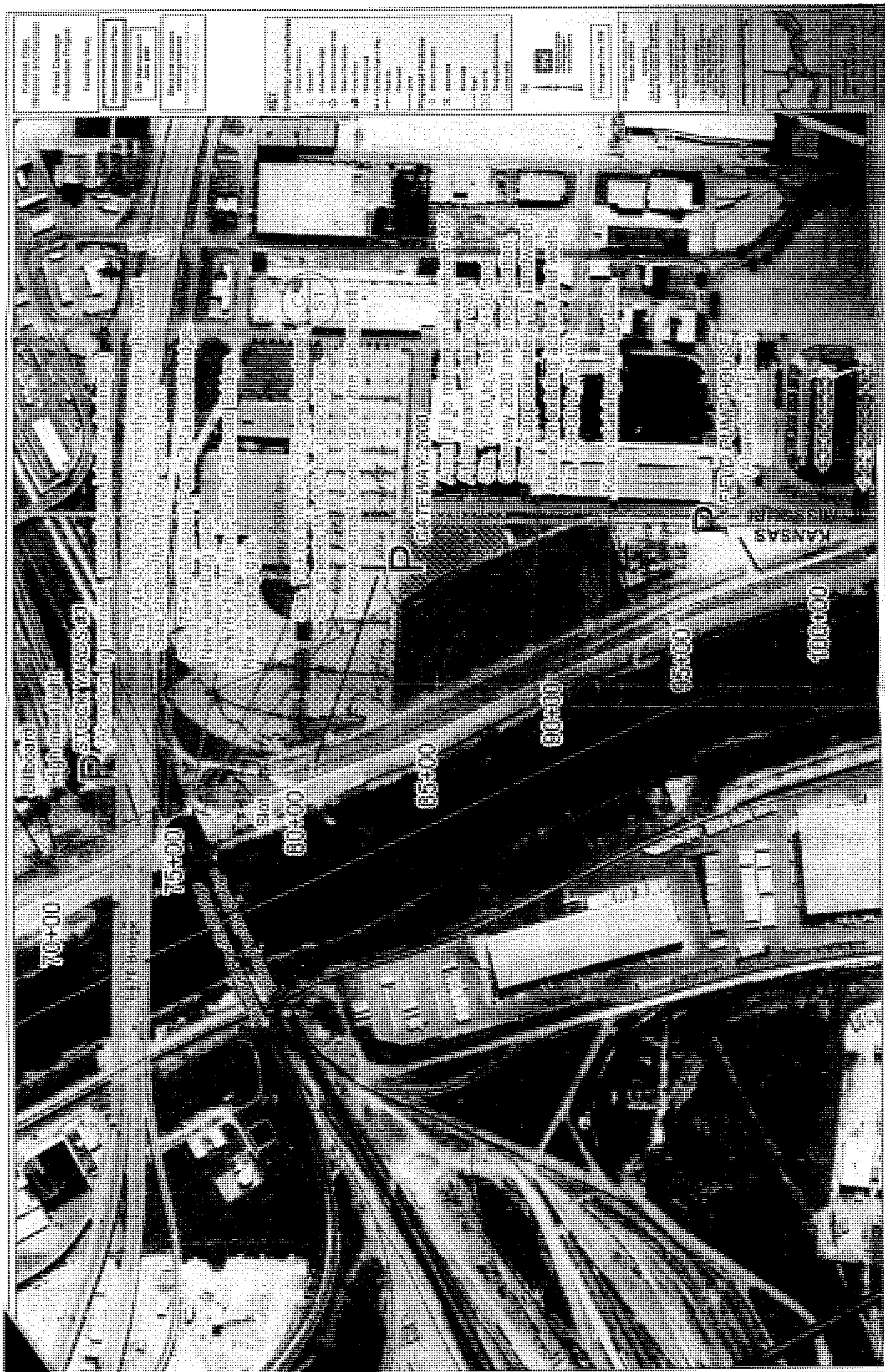
$$= 45\ yd^3\,(8) = \underline{360\ yd^3}$$

Total back fill for new inlets

$$\frac{55\,(9)}{27} = 20\ yd^3$$

$$2\,\frac{(12.5)(5)}{27} = 5\ yd^3$$

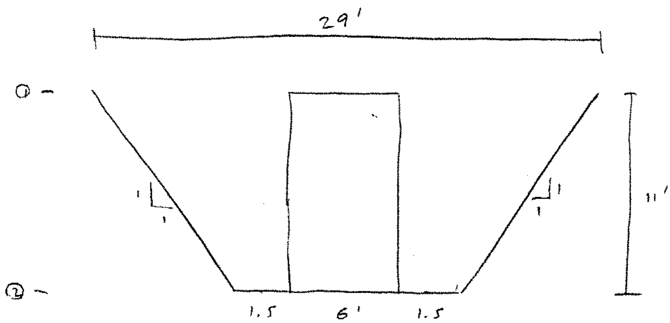
$$= 25\ yd^3\,(8) = \underline{200\ yd^3}$$


70+00
75+00
80+00
85+00
90+00
95+00
100+00

Approx 68+00 to 82+00

Replace MH 6 & 7

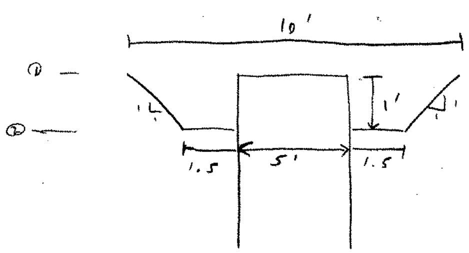
Raise MH 1, 2c, 5, and 4



① Area $= \frac{\pi}{4} 29^2 = 660.5 \ ft^2$

② Area $= \frac{\pi}{4} 9^2 = 63.6 \ ft^2$

Volume $= \frac{(660.5 + 63.6)}{2} (11) = 3982.8 \ ft^3 \rightarrow 148 \ yd^3$



Area $= \frac{\pi}{4} 10^2 = 78.5 \ ft^2$

Area $= \frac{\pi}{4} 8^2 = 50.3 \ ft^2$

Volume $= \frac{(78.5 + 50.3)}{2} (1) = 64.4 \ ft^3 \rightarrow 2.5 \ yd^3$

7-93

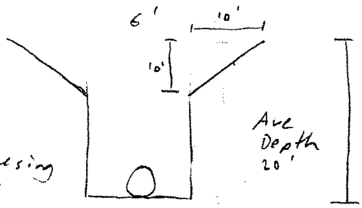
Quantity of Excavation
for replacement 148 yd^3 per MH

Quantity of Excavation
for raise = 2.5 yd^3

Approx 82+00 to Approx 94+50

Raise manholes 9, 10, and 11
Raise MH 9 – 6' Raise MH 10 – 5' Raise MH 11 – 1.5'
Quantity of excavation for each MH raise is 2.5 yd^3

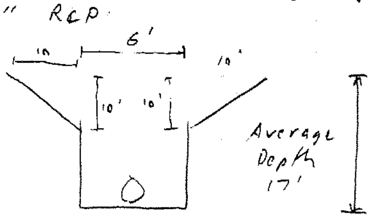
Replace 580 linear feet of 15" VCP with
15" RCP



Max
Depth
22'
Assume using
Trench Box
6x14 with
25' depth cut capability

Area = 220 ft^2
Length = 580'
Volume = 127600 ft^3
4726 yd^3 /1.8 = 15907

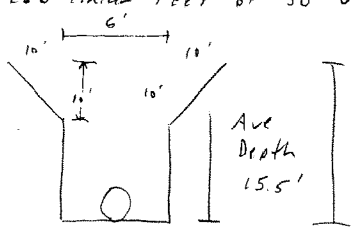
Replace 380 linear feet of 18" VCP with
18" RCP



Max
Depth
18'

Area = 1202 ft^2
Length = 380
Volume = 76,780 ft^3
= 2843 yd^3

Replace 260 linear feet of 30" VCP with 30" RCP



Max
Depth
16'

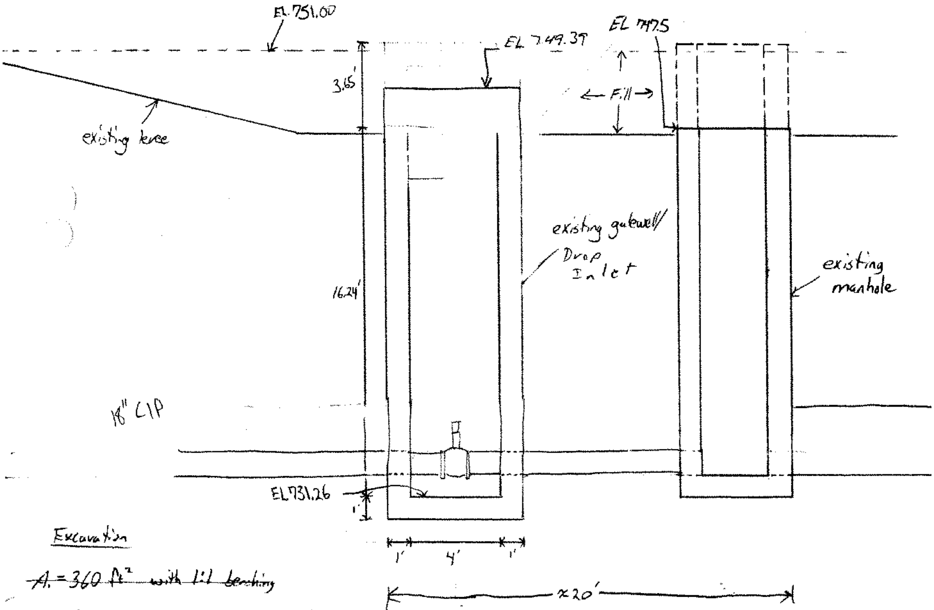
Area = 193 ft^2
Length = 260
Volume = 50180 ft^3
= 1859 yd^3

67+65
UL-27

- - - - - New fill Elevation
— — — — New structure

Assumptions

1. No asphalt or concrete pavement adjacent to gatewell.
2. Raise existing MH 4'
3. See structural Evaluation for existing Gatewell / Drop Inlet



Excavation

~~A_1 = 360 ft² with 1:1 benching~~

~~A = 616 ft² with 2:1 benching~~

15,080 ~~ft³ with 1:1~~ SEE BOOKS AVG
~~V = 8,636 ft³ with 2:1~~

~~Include 35' of shoring to prevent excavation into toe of levee~~ ~~- No SHORING INCLUDED IN THE LOCK, EXCAVATION WILL REMOVE SOME OF THE LEVEE~~

~~Assuming excavation continues 15 ft past drop inlet and manhole~~

CID-KS Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
80+54.12 MO	300	300	34	34	34	56	0.9	0.84	0	0	33.53	0	40	756	796	796	0.85	0.85	0.85	0.03	756	756	0	0.000
28+00 KS	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

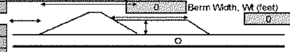
Lr 0 | Levee Width, L2 40

Levee Elev. 760.9 | Berm Width, Wt (feet) 0 | Berm Height, t (feet) 0

Ground Elev. 760

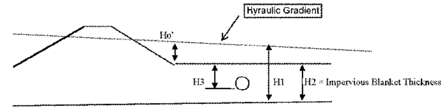
Blanket Base Elev. 726

Bedrock Elev. 670



9	Pipe Dept (ft)
751	Pipe Elev.
12	Pipe Diameter (in)
ML	Soil Type
115	Soil Unit Weight, γsoil (pcf)
9	Pipe Depth (Check)

All elevations are based on NGVD of 1929.



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	0.85	751	726	760	9	34	34.9	575.7	50	49	920	576	0	1.77	1.68
25	0.83	751	726	760	9	34	34.8	575.3	50	49	920	575	0	1.77	1.69
50	0.80	751	726	760	9	34	34.8	574.8	50	49	920	575	0	1.77	1.69
100	0.75	751	726	760	9	34	34.7	574.0	50	49	920	574	0	1.77	1.69
125	0.72	751	726	760	9	34	34.7	573.6	50	49	920	574	0	1.78	1.69
150	0.70	751	726	760	9	34	34.7	573.2	50	49	920	573	0	1.78	1.69
200	0.66	751	726	760	9	34	34.7	572.4	50	49	920	572	0	1.78	1.69
300	0.57	751	726	760	9	34	34.6	571.1	50	49	920	571	0	1.78	1.70
400	0.50	751	726	760	9	34	34.5	569.9	50	49	920	570	0	1.79	1.70
500	0.44	751	726	760	9	34	34.4	568.9	50	49	920	569	0	1.79	1.70

Ws Calc 12 -in steel 1lf
49.56
Wc Calc 49

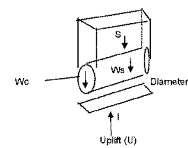
Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

Ground Elev 760 | Distance from toe (ft) 0

Ws = 50 | Wc = 49 Full

Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	760	0		50	49			
9	751	920	576	50	49	1.77	1.68	-336.27
10	750	1035	640	50	49	1.77	1.70	
11	749	1150	704	50	49	1.77	1.70	
12	748	1265	768	50	49	1.78	1.71	
13	747	1380	832	50	49	1.78	1.72	
14	746	1495	896	50	49	1.78	1.72	
15	745	1610	960	50	49	1.78	1.73	
16	744	1725	1023	50	49	1.78	1.73	
17	743	1840	1087	50	49	1.78	1.74	

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = based on 12-inch Nom Diam Steel Pipe, OD 12.75" wall thickness .375"
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((6/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot



U = P3 * diameter * l
Using l = 1

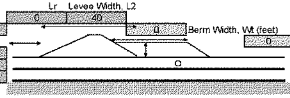
Conclusions
For levee stretch: 80+54.12 Mto 28+00 KS
For pipe diam (in): 12
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

CID-KS Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
80+54.12 MO	300	300	34	34	34	56	0.9	0.84	0	0	33.53	0	40	756	796	796	0.85	0.85	0.85	0.03	756	756	0	0.000
28+00 KS	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr Levee Width, L2

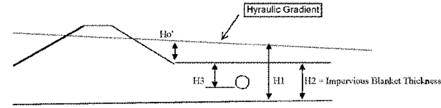
Levee Elev.	760.9	
Ground Elev.	760	
Blanket Base Elev.	726	
Bedrock Elev.	670	

Lr	L2	Berm Width, Wt (feet)	Berm Height, t (feet)
0	40	0	0



3	Pipe Dept (ft)
757	Pipe Elev.
12	Pipe Diameter (in)
ML	Soil Type
115	Soil Unit Weight, γsoil (pcf)
3	Pipe Depth (Check)

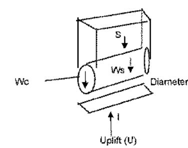
Depth Unknown assume 3'



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	0.85	757	726	760	3	34	34.9	191.9	50	49	230	192	0	1.71	1.46
25	0.83	757	726	760	3	34	34.8	191.8	50	49	230	192	0	1.71	1.46
50	0.80	757	726	760	3	34	34.8	191.6	50	49	230	192	0	1.71	1.46
100	0.75	757	726	760	3	34	34.7	191.3	50	49	230	191	0	1.72	1.46
125	0.72	757	726	760	3	34	34.7	191.2	50	49	230	191	0	1.72	1.46
150	0.70	757	726	760	3	34	34.7	191.1	50	49	230	191	0	1.72	1.46
200	0.66	757	726	760	3	34	34.7	190.8	50	49	230	191	0	1.72	1.47
300	0.57	757	726	760	3	34	34.6	190.4	50	49	230	190	0	1.73	1.47
400	0.50	757	726	760	3	34	34.5	190.0	50	49	230	190	0	1.73	1.47
500	0.44	757	726	760	3	34	34.4	189.6	50	49	230	190	0	1.73	1.47

Ws Calc 12 -in steel 1lf
49.56
Wc Calc 49

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 62.58 lb per ft (16-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((6/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev	760	Distance from toe (ft)	0		Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 50		Wc = 49 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	760	0		50	49		
3	757	230	192	50	49	1.71	1.46
4	756	345	256	50	49	1.73	1.54
5	755	460	320	50	49	1.75	1.59
6	754	575	384	50	49	1.75	1.63
7	753	690	448	50	49	1.76	1.65
8	752	805	512	50	49	1.77	1.67
9	751	920	576	50	49	1.77	1.68
10	750	1035	640	50	49	1.77	1.70
11	749	1150	704	50	49	1.77	1.70

Conclusions
For levee stretch: 80+54.12 Mto 28+00 KS
For pipe diam (in): 12
There is no uplift concern at a depth of 10 feet or deeper within 500 feet landward of the leve

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Lc	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
80+54.12 MO	300	300	34	34	34	56	0.9	0.84	0	0	33.53	0	40	756	796	796	0.85	0.85	0.85	0.03	756	756	0	0.000

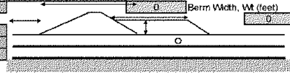
28+00 KS
500-yr + 3ft

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.

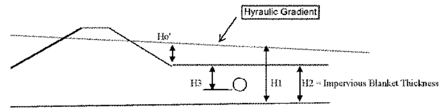
Lr 0 — Levee Width, L2 40

Levee Elev. 760.9 — Berm Width, Wt (feet) 0 — Berm Height, t (feet) 0

Ground Elev. 760
Blanket Base Elev. 726
Bedrock Elev. 670



3	Pipe Dept (ft)
757	Pipe Elev.
12	Pipe Diameter (in)
ML	Soil Type
115	Soil Unit Weight, γsoil (pcf)
3	Pipe Depth (Check)



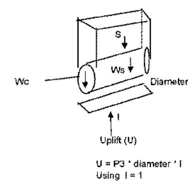
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	0.85	757	726	760	3	34	34.9	191.9	50	49	230	192	0	1.71	1.46
25	0.83	757	726	760	3	34	34.8	191.8	50	49	230	192	0	1.71	1.46
50	0.80	757	726	760	3	34	34.8	191.6	50	49	230	192	0	1.71	1.46
100	0.75	757	726	760	3	34	34.7	191.3	50	49	230	191	0	1.72	1.46
125	0.72	757	726	760	3	34	34.7	191.2	50	49	230	191	0	1.72	1.46
150	0.70	757	726	760	3	34	34.7	191.1	50	49	230	191	0	1.72	1.46
200	0.66	757	726	760	3	34	34.7	190.8	50	49	230	191	0	1.72	1.47
300	0.57	757	726	760	3	34	34.6	190.4	50	49	230	190	0	1.73	1.47
400	0.50	757	726	760	3	34	34.5	190.0	50	49	230	190	0	1.73	1.47
500	0.44	757	726	760	3	34	34.4	189.6	50	49	230	190	0	1.73	1.47

Ws Calc
12 -in steel 1lf
49.56
Wc Calc
49

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 760		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 50		Wc = 49 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	760	0		50	49			
3	757	230	192	50	49	1.71	1.46	-68.47
4	756	345	256	50	49	1.73	1.54	
5	755	460	320	50	49	1.75	1.59	
6	754	575	384	50	49	1.75	1.63	
7	753	690	448	50	49	1.76	1.65	
8	752	805	512	50	49	1.77	1.67	
9	751	920	576	50	49	1.77	1.68	
10	750	1035	640	50	49	1.77	1.70	
11	749	1150	704	50	49	1.77	1.70	

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 62.58 lb per ft (16-inch Diameter Steel Pipe, .375" thick)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((8/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot



Conclusions
For levee stretch: 80+54.12 ft to 28+00 KS
For pipe diam (in): 12
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

CID-KS Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
80+54.12 MO	300	300	34	34	34	56	0.9	0.84	0	0	33.53	0	40	756	796	796	0.85	0.85	0.85	0.03	756	756	0	0.000
28+00 KS	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 0 | Levee Width, L2 40

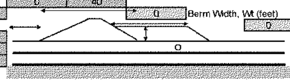
Levee Elev. 760.9 | Berm Width, Wt (feet) 0

Berm Height, t (feet) 0

Ground Elev. 760

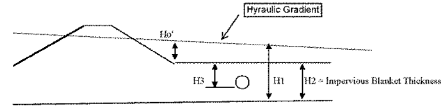
Blanket Base Elev. 726

Bedrock Elev. 670



7	Pipe Dept (ft)
753	Pipe Elev.
3	Pipe Diameter (in)
ML	Soil Type
115	Soil Unit Weight, γsoil (pcf)
7	Pipe Depth (Check)

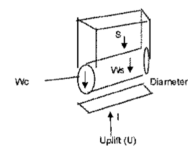
All elevations are based on NGVD of 1929.



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe ############	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	0.85	753	726	760	7	34	34.9	447.8	9	3	194	112	0	1.84	1.82
25	0.83	753	726	760	7	34	34.8	447.4	9	3	194	112	0	1.84	1.82
50	0.80	753	726	760	7	34	34.8	447.1	9	3	194	112	0	1.85	1.82
100	0.75	753	726	760	7	34	34.7	446.4	9	3	194	112	0	1.85	1.82
125	0.72	753	726	760	7	34	34.7	446.1	9	3	194	112	0	1.85	1.82
150	0.70	753	726	760	7	34	34.7	445.8	9	3	194	111	0	1.85	1.82
200	0.66	753	726	760	7	34	34.7	445.2	9	3	194	111	0	1.85	1.83
300	0.57	753	726	760	7	34	34.6	444.2	9	3	194	111	0	1.86	1.83
400	0.50	753	726	760	7	34	34.5	443.3	9	3	194	111	0	1.86	1.83
500	0.44	753	726	760	7	34	34.4	442.5	9	3	194	111	0	1.87	1.84

Ws Calc
3-in concrete 1lf
9.2039
Wc Calc
3.06

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length (3-inch Internal Diam Concrete, Class 2 Nonreinforced wall thick .75")
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((3/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



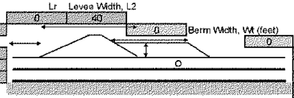
U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

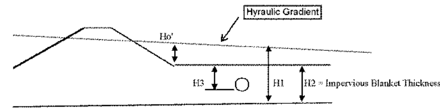
Ground Elev 760		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)	
	Ws = 9		Wc = 3 Full			
Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
760	0		9	3		
753	194	112	9	3	1.84	1.82
752	223	128	9	3	1.84	1.81
751	252	144	9	3	1.83	1.81
750	280	160	9	3	1.83	1.81
749	309	176	9	3	1.83	1.81
748	338	192	9	3	1.82	1.81
747	367	208	9	3	1.82	1.81
746	395	224	9	3	1.82	1.81
745	424	240	9	3	1.82	1.81

Conclusions
For levee stretch: 80+54.12 Mto 28+00 KS
For pipe diam (in): 3
There is no uplift concern at a depth of 40 inches (3.3 feet) or greater within 500 feet landward of the levee

CID-KS Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
80+54.12 MO	300	300	34	34	34	56	0.9	0.84	0	0	33.53	0	40	756	796	796	0.85	0.85	0.85	0.03	756	756	0	0.000
28+00 KS	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								



All elevations are based on NGVD of 1929.



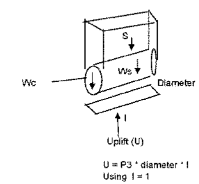
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 66 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	0.85	748.5	726	760	11.5	34	34.9	735.6	1542	1483	3795	4046	0	1.69	1.32
25	0.83	748.5	726	760	11.5	34	34.8	735.1	1542	1483	3795	4043	0	1.69	1.32
50	0.80	748.5	726	760	11.5	34	34.8	734.5	1542	1483	3795	4040	0	1.69	1.32
100	0.75	748.5	726	760	11.5	34	34.7	733.4	1542	1483	3795	4034	0	1.69	1.32
125	0.72	748.5	726	760	11.5	34	34.7	732.9	1542	1483	3795	4031	0	1.69	1.32
150	0.70	748.5	726	760	11.5	34	34.7	732.4	1542	1483	3795	4028	0	1.69	1.32
200	0.66	748.5	726	760	11.5	34	34.7	731.4	1542	1483	3795	4023	0	1.70	1.33
300	0.57	748.5	726	760	11.5	34	34.6	729.7	1542	1483	3795	4014	0	1.70	1.33
400	0.50	748.5	726	760	11.5	34	34.5	728.2	1542	1483	3795	4005	0	1.70	1.33
500	0.44	748.5	726	760	11.5	34	34.4	726.9	1542	1483	3795	3998	0	1.71	1.33

Ws Calc
66 -in steel 1lf
1542
Wc Calc
1483

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 1542 lb/ft (66-inch Diameter, Class B, based on Concrete Pipe Design Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((33/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

Ground Elev	760	Distance from toe (ft)	0			
	Ws = 1542		Wc = 1483 Full		Uplift Safety Extreme Case (1.1 REQ'D)	
Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
760	0		1542	1483		
748.5	3795	4046	1542	1483	1.69	1.32
747.5	4428	4398	1542	1483	1.69	1.36
746.5	5060	4750	1542	1483	1.70	1.39
745.5	5693	5102	1542	1483	1.71	1.42
744.5	6325	5453	1542	1483	1.71	1.44
743.5	6958	5805	1542	1483	1.72	1.46
742.5	7590	6157	1542	1483	1.72	1.48
741.5	8223	6509	1542	1483	1.73	1.50
740.5	8855	6861	1542	1483	1.73	1.52



Conclusions
For levee stretch: 80+54.12 Mto 28+00 KS
For pipe diam (in): 66
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 200 feet landward of the levee
There is uplift concern at a depth of 4.33 feet and less at the landward toe

CID-KS Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range 80+54.12 MO	300	300	34	34	34	56	0.9	0.84	0	0	33.53	0	40	756	796	796	0.85	0.85	0.85	0.03	756	756	0	0.000

28+00 KS
500-yr + 3ft

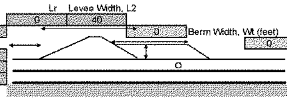
Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.

Lr 0 | Levee Width, L2 40

Levee Elev. 760.9

Berm Width, Wt (feet) 0

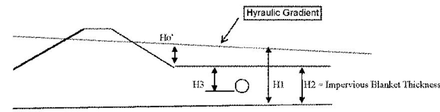
Berm Height, t (feet) 0



Ground Elev.	760
Blanket Base Elev.	726
Bedrock Elev.	670

3	Pipe Dept (ft)
757	Pipe Elev.
8	Pipe Diameter (in)
ML	Soil Type
115	Soil Unit Weight, γsoil (pcf)
3	Pipe Depth (Check)

All elevations are based on NGVD of 1929.



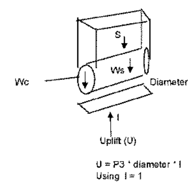
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 8 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	0.85	757	726	760	3	34	34.9	191.9	29	22	179	128	0	1.79	1.62
25	0.83	757	726	760	3	34	34.8	191.8	29	22	179	128	0	1.79	1.62
50	0.80	757	726	760	3	34	34.8	191.6	29	22	179	128	0	1.79	1.62
100	0.75	757	726	760	3	34	34.7	191.3	29	22	179	128	0	1.80	1.63
125	0.72	757	726	760	3	34	34.7	191.2	29	22	179	127	0	1.80	1.63
150	0.70	757	726	760	3	34	34.7	191.1	29	22	179	127	0	1.80	1.63
200	0.66	757	726	760	3	34	34.7	190.8	29	22	179	127	0	1.80	1.63
300	0.57	757	726	760	3	34	34.6	190.4	29	22	179	127	0	1.81	1.63
400	0.50	757	726	760	3	34	34.5	190.0	29	22	179	127	0	1.81	1.64
500	0.44	757	726	760	3	34	34.4	189.6	29	22	179	126	0	1.81	1.64

Ws Calc
8 -in steel 1lf
28.55
Wc Calc
22

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = based on 8-inch Nom Diam Steel Pipe, OD 8.625" wall thickness .322")
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((4/12)^2)*1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 760	Distance from toe (ft) 0				Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 29		Wc = 22 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	760	0		29	22			
3	757	179	128	29	22	1.79	1.62	-66.71
4	756	256	171	29	22	1.79	1.67	
5	755	332	213	29	22	1.79	1.69	
6	754	409	256	29	22	1.79	1.71	
7	753	486	299	29	22	1.80	1.72	
8	752	562	341	29	22	1.80	1.73	
9	751	639	384	29	22	1.80	1.74	
10	750	716	426	29	22	1.80	1.74	
11	749	792	469	29	22	1.80	1.75	



Conclusions
For levee stretch: 80+54.12 Mto 28+00 KS
For pipe diam (in): 8
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

CID-KS Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L1																							feet	
Stationing Range																								
80+54.12 MO	300	300	34	34	34	56	0.9	0.84	0	0	33.53	0	40	756	796	796	0.85	0.85	0.85	0.03	756	756	0	0.000
28+00 KS	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 0 | Levee Width, L2 40

Levee Elev. 760.9

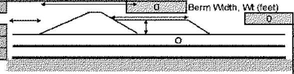
Berm Width, Wt (feet) 0

Berm Height, t (feet) 0

Ground Elev. 760

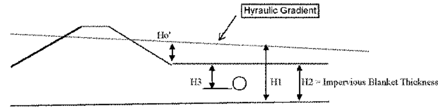
Blanket Base Elev. 726

Bedrock Elev. 670



3	Pipe Dept (ft)
757	Pipe Elev.
3	Pipe Diameter (in)
ML	Soil Type
115	Soil Unit Weight, γsoil (pcf)
3	Pipe Depth (Check)

All elevations are based on NGVD of 1929.



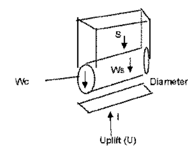
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 3 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	0.85	757	726	760	3	34	34.9	191.9	8	3	79	48	0	1.87	1.81
25	0.83	757	726	760	3	34	34.8	191.8	8	3	79	48	0	1.87	1.81
50	0.80	757	726	760	3	34	34.8	191.6	8	3	79	48	0	1.87	1.81
100	0.75	757	726	760	3	34	34.7	191.3	8	3	79	48	0	1.88	1.81
125	0.72	757	726	760	3	34	34.7	191.2	8	3	79	48	0	1.88	1.81
150	0.70	757	726	760	3	34	34.7	191.1	8	3	79	48	0	1.88	1.81
200	0.66	757	726	760	3	34	34.7	190.8	8	3	79	48	0	1.88	1.82
300	0.57	757	726	760	3	34	34.6	190.4	8	3	79	48	0	1.88	1.82
400	0.50	757	726	760	3	34	34.5	190.0	8	3	79	47	0	1.89	1.82
500	0.44	757	726	760	3	34	34.4	189.6	8	3	79	47	0	1.89	1.83

Ws Calc
3 -in steel 1lf
7.58
Wc Calc
3

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = based on 3-inch Nom Diam Steel Pipe, OD 3.5" wall thickness .216")
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((1.5/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

Ground Elev 760		Distance from toe (ft) 0				
	Ws = 8		Wc = 3 Full		Uplift Safety Extreme Case (1.1 REQ'D)	
Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
760	0		8	3		
757	79	48	8	3	1.87	1.81
756	108	64	8	3	1.85	1.80
755	137	80	8	3	1.84	1.80
754	165	96	8	3	1.83	1.80
753	194	112	8	3	1.83	1.80
752	223	128	8	3	1.82	1.80
751	252	144	8	3	1.82	1.80
750	280	160	8	3	1.82	1.80
749	309	176	8	3	1.82	1.80



U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 80+54.12 f to 28+00 KS
For pipe diam (in): 3
There is no uplift concern at a depth of 40 inches (3.3 feet) or greater within 500 feet landward of the leve

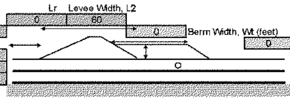
CID-KS Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L3																							feet	
Stationing Range																								
38+00	400	400	31	31	31	50	12	0.84	0	0	2.34	0	60	787	847	847	11.15	11.15	11.15	0.36	787	787	0	0.006
63+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Levee Elev. 763

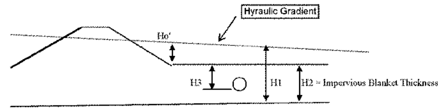
Ground Elev. 751

Blanket Base Elev. 720

Bedrock Elev. 670



20	Pipe Dept (ft)
731	Pipe Elev.
36	Pipe Diameter (in)
ML-CL	Soil Type
115	Soil Unit Weight, γsoil (pcf)
20	Pipe Depth (Check)



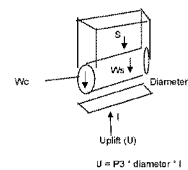
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 36 -in RCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	11.15	731	720	751	20	31	42.2	1696.9	524	441	5865	5091	0	1.34	1.26
25	10.80	731	720	751	20	31	41.8	1682.9	524	441	5865	5049	0	1.35	1.27
50	10.46	731	720	751	20	31	41.5	1669.3	524	441	5865	5008	0	1.36	1.28
100	9.82	731	720	751	20	31	40.8	1643.4	524	441	5865	4930	0	1.39	1.30
125	9.51	731	720	751	20	31	40.5	1631.0	524	441	5865	4893	0	1.40	1.31
150	9.22	731	720	751	20	31	40.2	1619.0	524	441	5865	4857	0	1.41	1.32
200	8.65	731	720	751	20	31	39.6	1596.2	524	441	5865	4789	0	1.43	1.33
300	7.62	731	720	751	20	31	38.6	1554.7	524	441	5865	4684	0	1.46	1.37
400	6.71	731	720	751	20	31	37.7	1518.1	524	441	5865	4554	0	1.50	1.40
500	5.91	731	720	751	20	31	36.9	1485.9	524	441	5865	4458	0	1.53	1.43

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 751		Distance from toe (ft) 0					
		Ws = 524		Wc = 441 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	751	0		524	441			
20	731	5865	5091	524	441	1.34	1.26	-789.26
21	730	6210	5345	524	441	1.34	1.26	
22	729	6555	5600	524	441	1.34	1.26	
23	728	6900	5854	524	441	1.34	1.27	
24	727	7245	6109	524	441	1.34	1.27	
25	726	7590	6363	524	441	1.34	1.28	
26	725	7935	6618	524	441	1.34	1.28	
27	724	8280	6872	524	441	1.35	1.28	
28	723	8625	7127	524	441	1.35	1.28	

Ws Calc
36 -in RCP 1lf
524
Wc Calc
441

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 524 lb/ft (36-inch Diameter, Class B, based on Concrete Pipe Design Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((18/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot



Conclusions
For levee stretch: 38+00 to 63+00
For pipe diam (in): 36

CID-KS Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L3																							feet	
Stationing Range																								
38+00	400	400	31	31	31	50	12	0.84	0	0	2.34	0	60	787	847	847	11.15	11.15	11.15	0.36	787	787	0	0.000
63+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 0 Levee Width, L2 60

Levee Elev. 763

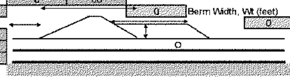
Berm Width, Wt (feet) 0

Berm Height, t (feet) 0

Ground Elev. 751

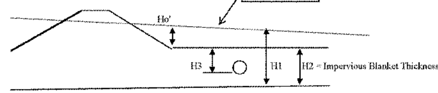
Blanket Base Elev. 720

Bedrock Elev. 670



3	Pipe Dept (ft)
748	Pipe Elev.
12	Pipe Diameter (in)
ML-CL	Soil Type
115	Soil Unit Weight, γsoil (pcf)
3	Pipe Depth (Check)

All elevations are based on NGVD of 1929.



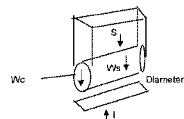
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	11.15	748	720	751	3	31	42.2	254.5	50	49	230	255	0	1.29	1.10
25	10.80	748	720	751	3	31	41.8	252.4	50	49	230	252	0	1.30	1.11
50	10.46	748	720	751	3	31	41.5	250.4	50	49	230	250	0	1.31	1.12
100	9.82	748	720	751	3	31	40.8	246.5	50	49	230	247	0	1.33	1.13
125	9.51	748	720	751	3	31	40.5	244.6	50	49	230	245	0	1.34	1.14
150	9.22	748	720	751	3	31	40.2	242.9	50	49	230	243	0	1.35	1.15
200	8.65	748	720	751	3	31	39.6	239.4	50	49	230	239	0	1.37	1.17
300	7.62	748	720	751	3	31	38.6	233.2	50	49	230	233	0	1.41	1.20
400	6.71	748	720	751	3	31	37.7	227.7	50	49	230	228	0	1.44	1.23
500	5.91	748	720	751	3	31	36.9	222.9	50	49	230	223	0	1.47	1.25

Ws Calc
12 -in steel 1lf
49.56
Wc Calc 49

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = based on 12-inch Nom Diam Steel Pipe, OD 12.75" wall thickness .375"
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((6/12)^2) *1*62.4 = 49 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
Ground Elev 751		Ws = 50	Distance from toe (ft) 50	Wc = 49 Full		Uplift Safety Extreme Case (1.1 REQ'D)		
	751	0		50	49			
3	748	230	250	50	49	1.31	1.12	-4.13
4	747	345	334	50	49	1.33	1.18	
5	746	460	417	50	49	1.34	1.22	
6	745	575	501	50	49	1.35	1.25	
7	744	690	584	50	49	1.35	1.27	
8	743	805	668	50	49	1.35	1.28	
9	742	920	751	50	49	1.36	1.29	
10	741	1035	835	50	49	1.36	1.30	
11	740	1150	918	50	49	1.36	1.31	



U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 38+00 to 63+00
For pipe diam (in): 12
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

CID-KS Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L3																							feet	
Stationing Range																								
38+00	400	400	31	31	31	50	12	0.84	0	0	2.34	0	60	787	847	847	11.15	11.15	11.15	0.36	787	787	0	0.000
63+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 0 Levee Width, L2 60

Levee Elev. 763 — 0 Berm Width, Wt (feet)

0 Berm Height, t (feet)

Ground Elev. 751 — 3 Pipe Dept (ft)

Blanket Base Elev. 720 — 748 Pipe Elev.

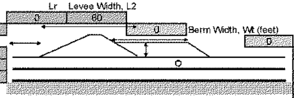
Bedrock Elev. 670 — 8 Pipe Diameter (in)

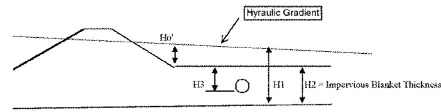
ML-CL Soil Type

115 Soil Unit Weight, γsoil (pcf)

3 Pipe Depth (Check)

All elevations are based on NGVD of 1929.





d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 8 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	11.15	748	720	751	3	31	42.2	254.5	29	22	179	170	0	1.35	1.22
25	10.80	748	720	751	3	31	41.8	252.4	29	22	179	168	0	1.36	1.23
50	10.46	748	720	751	3	31	41.5	250.4	29	22	179	167	0	1.37	1.24
100	9.82	748	720	751	3	31	40.8	246.5	29	22	179	164	0	1.39	1.26
125	9.51	748	720	751	3	31	40.5	244.6	29	22	179	163	0	1.41	1.27
150	9.22	748	720	751	3	31	40.2	242.9	29	22	179	162	0	1.42	1.28
200	8.65	748	720	751	3	31	39.6	239.4	29	22	179	160	0	1.44	1.30
300	7.62	748	720	751	3	31	38.6	233.2	29	22	179	155	0	1.47	1.33
400	6.71	748	720	751	3	31	37.7	227.7	29	22	179	152	0	1.51	1.37
500	5.91	748	720	751	3	31	36.9	222.9	29	22	179	149	0	1.54	1.40

Ws Calc
8 -in steel 1lf
28.55
Wc Calc
22

See Sample Calculations for list of abbreviations and sample calculations

Ws = weight of structure per foot of length = based on 12-inch Nom Diam Steel Pipe, OD 8.625" wall thickness .322")

Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((8/12)^2) *1*62.4 = 49 pl

S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil

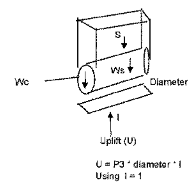
P3 = H3*(H1/H2)*γwater

U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3

Wg = weight of surcharge water above top surface of structure control by gravity flow

SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Ca = Weight of concrete anchor needed to prevent uplift per linear foot



Use this table to determine the distance from toe at which the Safety Factor is me

Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 751		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 29		Wc = 22 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	751	0		29	22			
3	748	179	170	29	22	1.35	1.22	-20.78
4	747	256	226	29	22	1.35	1.26	
5	746	332	283	29	22	1.35	1.28	
6	745	409	339	29	22	1.35	1.29	
7	744	486	396	29	22	1.35	1.30	
8	743	562	453	29	22	1.35	1.31	
9	742	639	509	29	22	1.35	1.31	
10	741	716	566	29	22	1.35	1.32	
11	740	792	622	29	22	1.35	1.32	

Conclusions

For levee stretch: 38+00 to 63+00

For pipe diam (in): 8

There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee

There is uplift concern at a depth of 8.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L4																							feet	
Stationing Range																								
63+00	400	400	31	31	31	50	14	0.84	0	0	2.10	0	100	787	887	887	12.42	12.42	12.42	0.40	787	787	0	0.000
74+75	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 0 — Levee Width, L2 100

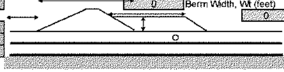
Levee Elev. 765 — Berm Width, Wt (feet) 0

Berm Height, t (feet) 0

Ground Elev. 751

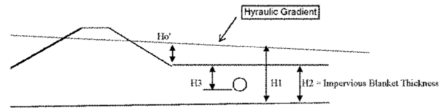
Blanket Base Elev. 720

Bedrock Elev. 670



15	Pipe Dept (ft)
736	Pipe Elev.
12	Pipe Diameter (in)
ML-CL	Soil Type
115	Soil Unit Weight, γsoil (pcf)
15	Pipe Depth (Check)

12" Pipe Entering MH 1



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in CIP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	12.42	736	720	751	15	31	43.4	1311.1	93	49	1610	1311	0	1.34	1.30
25	12.03	736	720	751	15	31	43.0	1299.4	93	49	1610	1299	0	1.35	1.31
50	11.66	736	720	751	15	31	42.7	1288.0	93	49	1610	1288	0	1.36	1.32
100	10.94	736	720	751	15	31	41.9	1266.3	93	49	1610	1266	0	1.38	1.34
125	10.60	736	720	751	15	31	41.6	1256.0	93	49	1610	1256	0	1.39	1.36
150	10.27	736	720	751	15	31	41.3	1246.0	93	49	1610	1246	0	1.41	1.37
200	9.64	736	720	751	15	31	40.6	1226.9	93	49	1610	1227	0	1.43	1.39
300	8.49	736	720	751	15	31	39.5	1192.2	93	49	1610	1192	0	1.47	1.43
400	7.47	736	720	751	15	31	38.5	1161.7	93	49	1610	1162	0	1.51	1.47
500	6.58	736	720	751	15	31	37.6	1134.8	93	49	1610	1135	0	1.54	1.50

Ws Calc 12 -in CIP 1lf
93
Wc Calc 49

See Sample Calculations for list of abbreviations and sample calculations

Ws = weight of structure per foot of length 93 lb/ft (12-inch Diameter, Class B, based on Concrete Pipe Design Manual)

Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((16/12)^2) *1*62.4

S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diamater/12)*1*γsoil

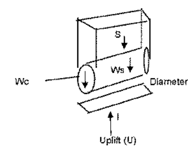
P3 = H3*(H1/H2)*γwater

U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3

Wg = weight of surcharge water above top surface of structure control by gravity flow

SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Ca = Weight of concrete anchor needed to prevent uplift per linear foot



U = P3 * diameter * l

Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is me

Enter the Distance from toe and change the value until the SFf is equal to that required

Depth Below Ground	Ground Elev 751 Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Distance from toe (ft) 0 Uplift (U) Force, lbs/ft	Ws = 93 Pipe Weight, Lbs/ft	Wc = 49 Full water Wt, Lbs / ft	Uplift Safety Extreme Case (1.1 REQ'D) SFf Full	SFf Empty	Ca Lbs/ft
	751	0		93	49			
15	736	1610	1311	93	49	1.34	1.30	-260.82
16	735	1725	1398	93	49	1.34	1.30	
17	734	1840	1486	93	49	1.33	1.30	
18	733	1955	1573	93	49	1.33	1.30	
19	732	2070	1661	93	49	1.33	1.30	
20	731	2185	1748	93	49	1.33	1.30	
21	730	2300	1836	93	49	1.33	1.30	
22	729	2415	1923	93	49	1.33	1.30	
23	728	2530	2010	93	49	1.33	1.30	

Conclusions

For levee stretch: 63+00 to 74+75

For pipe diam (in): 12

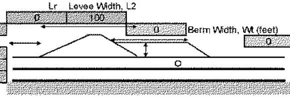
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee

There is uplift concern at a depth of 8.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L4 Stationing Range																							feet	
63+00	400	400	31	31	31	50	14	0.84	0	0	2.10	0	100	787	887	887	12.42	12.42	12.42	0.40	787	787	0	0.000
74+75 500-yr + 3ft	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							

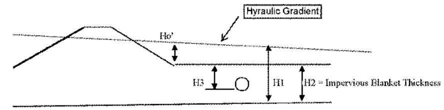
Lr Levee Width, L2: 0, 100

Levee Elev.	755
Ground Elev.	751
Blanket Base Elev.	720
Bedrock Elev.	670



0	Berm Width, Wt (feet)
0	Berm Height, t (feet)
15	Pipe Dept (ft)
736	Pipe Elev.
18	Pipe Diameter (in)
ML-CL	Soil Type
115	Soil Unit Weight, γsoil (pcf)
15	Pipe Depth (Check)

18" CIP Entering MH 2



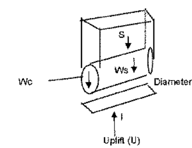
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 18 -in CIP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	12.42	736	720	751	15	31	43.4	1311.1	168	110	2329	1967	0	1.33	1.27
25	12.03	736	720	751	15	31	43.0	1299.4	168	110	2329	1949	0	1.34	1.28
50	11.66	736	720	751	15	31	42.7	1288.0	168	110	2329	1932	0	1.35	1.29
100	10.94	736	720	751	15	31	41.9	1266.3	168	110	2329	1900	0	1.37	1.31
125	10.60	736	720	751	15	31	41.6	1256.0	168	110	2329	1884	0	1.38	1.33
150	10.27	736	720	751	15	31	41.3	1246.0	168	110	2329	1869	0	1.39	1.34
200	9.64	736	720	751	15	31	40.6	1226.9	168	110	2329	1840	0	1.42	1.36
300	8.49	736	720	751	15	31	39.5	1192.2	168	110	2329	1788	0	1.46	1.40
400	7.47	736	720	751	15	31	38.5	1161.7	168	110	2329	1743	0	1.50	1.43
500	6.58	736	720	751	15	31	37.6	1134.8	168	110	2329	1702	0	1.53	1.47

Ws Calc 18 -in CIP 1lf
168
Wc Calc 110

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 168 lb/ft (18-inch Diameter, Class B, based on Concrete Pipe Design Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((18/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 751	Distance from toe (ft) 0				Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 168		Wc = 110 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	751	0		168	110			
15	736	2329	1967	168	110	1.33	1.27	-333.48
16	735	2501	2098	168	110	1.33	1.27	
17	734	2674	2229	168	110	1.32	1.27	
18	733	2846	2360	168	110	1.32	1.28	
19	732	3019	2491	168	110	1.32	1.28	
20	731	3191	2622	168	110	1.32	1.28	
21	730	3364	2753	168	110	1.32	1.28	
22	729	3536	2884	168	110	1.32	1.28	
23	728	3709	3015	168	110	1.32	1.29	



U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 63+00 to 74+75
For pipe diam (in): 18
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L4																							feet	
Stationing Range																								
63+00	400	400	31	31	31	50	14	0.84	0	0	2.10	0	100	787	887	887	12.42	12.42	12.42	0.40	787	787	0	0.000
74+75	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 0 — Levee Width, L2 100

Levee Elev. 765 — Berm Width, Wt (feet) 0

Berm Height, t (feet) 0

Ground Elev. 751 — Pipe Dept (ft) 9

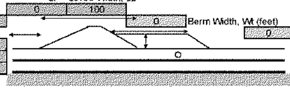
Blanket Base Elev. 720 — Pipe Elev. 742

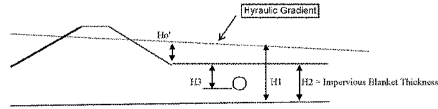
Bedrock Elev. 670 — Pipe Diameter (in) 30

Soil Type ML-CL

Soil Unit Weight, γsoil (pcf) 115

Pipe Depth (Check) 9





d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 30 -in RCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	12.42	742	720	751	9	31	43.4	786.6	384	306	1869	1967	0	1.30	1.15
25	12.03	742	720	751	9	31	43.0	779.6	384	306	1869	1949	0	1.31	1.16
50	11.66	742	720	751	9	31	42.7	772.8	384	306	1869	1932	0	1.32	1.17
100	10.94	742	720	751	9	31	41.9	759.8	384	306	1869	1900	0	1.35	1.19
125	10.60	742	720	751	9	31	41.6	753.6	384	306	1869	1884	0	1.36	1.20
150	10.27	742	720	751	9	31	41.3	747.6	384	306	1869	1869	0	1.37	1.21
200	9.64	742	720	751	9	31	40.6	736.2	384	306	1869	1840	0	1.39	1.22
300	8.49	742	720	751	9	31	39.5	715.3	384	306	1869	1788	0	1.43	1.26
400	7.47	742	720	751	9	31	38.5	697.0	384	306	1869	1743	0	1.47	1.29
500	6.58	742	720	751	9	31	37.6	680.9	384	306	1869	1702	0	1.50	1.32

Ws Calc
30 -in RCP 1lf
384

Wc Calc
306

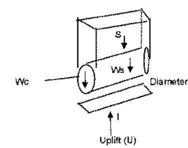
See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 384 lb/ft (30-inch Diameter, Class B, based on Concrete Pipe Design Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((15/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

Ground Elev 751 — Distance from toe (ft) 0

Ws = 384 — Wc = 306 Full

Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	Uplift Safety Extreme Case (1.1 REQ'D) SFf Full	SFf Empty	Ca Lbs/ft
	751	0		384	306			
9	742	1869	1967	384	306	1.30	1.15	-89.48
10	741	2156	2185	384	306	1.30	1.16	
11	740	2444	2404	384	306	1.30	1.18	
12	739	2731	2622	384	306	1.30	1.19	
13	738	3019	2841	384	306	1.31	1.20	
14	737	3306	3059	384	306	1.31	1.21	
15	736	3594	3278	384	306	1.31	1.21	
16	735	3881	3496	384	306	1.31	1.22	
17	734	4169	3715	384	306	1.31	1.23	



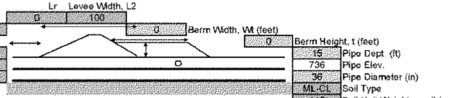
U = P3 * diameter * l
Using l = 1

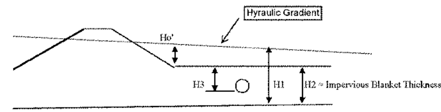
Conclusions
For levee stretch: 63+00 to 74+75
For pipe diam (in): 30
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L4																							feet	
Stationing Range																								
63+00	400	400	31	31	31	50	14	0.84	0	0	2.10	0	100	787	887	887	12.42	12.42	12.42	0.40	787	787	0	0.000
74+75	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr Levee Width, L2

Levee Elev.	765	
Ground Elev.	751	
Blanket Base Elev.	720	
Bedrock Elev.	670	

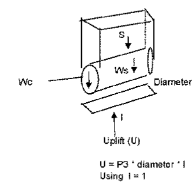




d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 36 -in RCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	12.42	736	720	751	15	31	43.4	1311.1	524	441	4140	3933	0	1.30	1.19
25	12.03	736	720	751	15	31	43.0	1299.4	524	441	4140	3898	0	1.31	1.20
50	11.66	736	720	751	15	31	42.7	1288.0	524	441	4140	3864	0	1.32	1.21
100	10.94	736	720	751	15	31	41.9	1266.3	524	441	4140	3799	0	1.34	1.23
125	10.60	736	720	751	15	31	41.6	1256.0	524	441	4140	3768	0	1.35	1.24
150	10.27	736	720	751	15	31	41.3	1246.0	524	441	4140	3738	0	1.37	1.25
200	9.64	736	720	751	15	31	40.6	1226.9	524	441	4140	3681	0	1.39	1.27
300	8.49	736	720	751	15	31	39.5	1192.2	524	441	4140	3577	0	1.43	1.30
400	7.47	736	720	751	15	31	38.5	1161.7	524	441	4140	3485	0	1.46	1.34
500	6.58	736	720	751	15	31	37.6	1134.8	524	441	4140	3404	0	1.50	1.37

Ws Calc 36 -in RCP 1lf
524
Wc Calc 441

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 524 lb/ft (36-inch Diameter, Class B, based on Concrete Pipe Design Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((18/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot

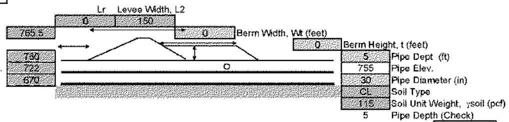


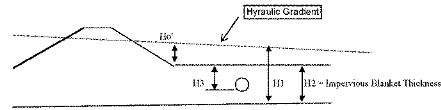
Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 751		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 524		Wc = 441 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	751	0		524	441			
15	736	4140	3933	524	441	1.30	1.19	-337.45
16	735	4485	4195	524	441	1.30	1.19	
17	734	4830	4458	524	441	1.30	1.20	
18	733	5175	4720	524	441	1.30	1.21	
19	732	5520	4982	524	441	1.30	1.21	
20	731	5865	5244	524	441	1.30	1.22	
21	730	6210	5507	524	441	1.30	1.22	
22	729	6555	5769	524	441	1.30	1.23	
23	728	6900	6031	524	441	1.30	1.23	

Conclusions
For levee stretch: 63+00 to 74+75
For pipe diam (in): 36
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L5																							feet	
Stationing Range																								
74+75	500	500	38	38	38	52	5.5	0.84	0	0	6.70	0	150	994	1144	1144	4.78	4.78	4.78	0.13	994	994	0	0.000
77+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

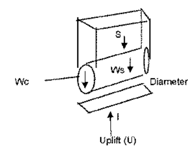




d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 30 -RCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	4.78	755	722	760	5	38	42.8	351.2	384	306	719	878	0	1.60	1.26
25	4.66	755	722	760	5	38	42.7	350.3	384	306	719	876	0	1.61	1.26
50	4.54	755	722	760	5	38	42.5	349.3	384	306	719	873	0	1.61	1.26
100	4.32	755	722	760	5	38	42.3	347.5	384	306	719	869	0	1.62	1.27
125	4.21	755	722	760	5	38	42.2	346.6	384	306	719	867	0	1.63	1.27
150	4.11	755	722	760	5	38	42.1	345.7	384	306	719	864	0	1.63	1.28
200	3.91	755	722	760	5	38	41.9	344.1	384	306	719	860	0	1.64	1.28
300	3.53	755	722	760	5	38	41.5	341.0	384	306	719	853	0	1.65	1.29
400	3.20	755	722	760	5	38	41.2	338.2	384	306	719	846	0	1.67	1.30
500	2.89	755	722	760	5	38	40.9	335.7	384	306	719	839	0	1.68	1.31

Ws Calc
30 -in RCP 1lf
384
Wc Calc
306

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 384 lb/ft (30-inch Diameter, Class B, based on Concrete Pipe Design Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((18/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot



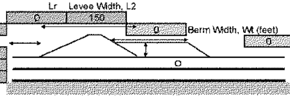
U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 760		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 384		Wc = 306 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	760	0		384	306			
5	755	719	878	384	306	1.60	1.26	-136.85
6	754	1006	1054	384	306	1.61	1.32	
7	753	1294	1229	384	306	1.61	1.36	
8	752	1581	1405	384	306	1.62	1.40	
9	751	1869	1581	384	306	1.62	1.43	
10	750	2156	1756	384	306	1.62	1.45	
11	749	2444	1932	384	306	1.62	1.46	
12	748	2731	2107	384	306	1.62	1.48	
13	747	3019	2283	384	306	1.62	1.49	

Conclusions
For levee stretch: 74+75 to 77+00
For pipe diam (in): 30
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L6																							feet	
Stationing Range																								
77+00	400	400	29	29	29	50	18	0.64	0	0	1.63	0	150	762	912	912	15.04	15.04	15.04	0.52	762	762	0	0.000
97+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								



Levee Elev. 767

Ground Elev. 749

Blanket Base Elev. 720

Bedrock Elev. 670

18.26 Pipe Dept (ft)

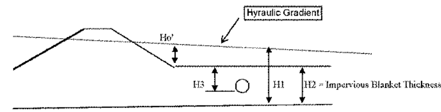
730.74 Pipe Elev.

12 Pipe Diameter (in)

ML-CL Soil Type

115 Soil Unit Weight, γsoil (pcf)

18.26 Pipe Depth (Check)



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in VCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	15.04	730.74	720	749	18.26	29	44.0	1730.3	54	49	1985	1730	0	1.21	1.18
25	14.55	730.74	720	749	18.26	29	43.6	1711.2	54	49	1985	1711	0	1.22	1.19
50	14.08	730.74	720	749	18.26	29	43.1	1692.7	54	49	1985	1693	0	1.23	1.20
100	13.19	730.74	720	749	18.26	29	42.2	1657.6	54	49	1985	1658	0	1.26	1.23
125	12.76	730.74	720	749	18.26	29	41.8	1640.8	54	49	1985	1641	0	1.27	1.24
150	12.35	730.74	720	749	18.26	29	41.3	1624.6	54	49	1985	1625	0	1.29	1.25
200	11.56	730.74	720	749	18.26	29	40.6	1593.8	54	49	1985	1594	0	1.31	1.28
300	10.14	730.74	720	749	18.26	29	39.1	1537.9	54	49	1985	1538	0	1.36	1.33
400	8.89	730.74	720	749	18.26	29	37.9	1488.9	54	49	1985	1489	0	1.40	1.37
500	7.80	730.74	720	749	18.26	29	36.8	1445.9	54	49	1985	1446	0	1.44	1.41

Ws Calc 12 -in VCP 1lf
54
Wc Calc 49

See Sample Calculations for list of abbreviations and sample calculations

Ws = weight of structure per foot of length 54 lb/ft (12-inch Diameter, Standard Bell & Spigot VCP Manual)

Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((13/12)^2) *1*62.4

S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil

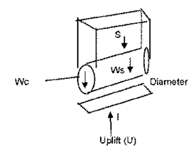
P3 = H3*(H1/H2)*γwater

U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3

Wg = weight of surcharge water above top surface of structure control by gravity flow

SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Ca = Weight of concrete anchor needed to prevent uplift per linear foot



U = P3 * diameter * l

Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is me

Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 749		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 54		Wc = 49 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	749	0		54	49			
18.26	730.74	1985	1730	54	49	1.21	1.18	-135.59
19.26	729.74	2100	1825	54	49	1.21	1.18	
20.26	728.74	2215	1920	54	49	1.21	1.18	
21.26	727.74	2330	2015	54	49	1.21	1.18	
22.26	726.74	2445	2109	54	49	1.21	1.18	
23.26	725.74	2560	2204	54	49	1.21	1.19	
24.26	724.74	2675	2299	54	49	1.21	1.19	
25.26	723.74	2790	2394	54	49	1.21	1.19	
26.26	722.74	2905	2488	54	49	1.21	1.19	

Conclusions

For levee stretch: 77+00 to 97+00

For pipe diam (in): 12

There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee

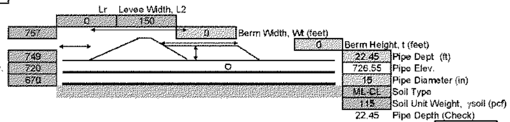
There is uplift concern at a depth of 8.33 feet and less at the landward toe

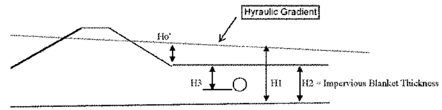
Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L6																							feet	
Stationing Range																								
77+00	400	400	29	29	29	50	18	0.84	0	0	1.63	0	150	762	912	912	15.04	15.04	15.04	0.52	762	762	0	0.000

97+00

500-yr + 3ft

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.





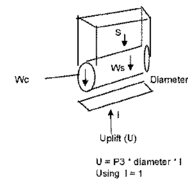
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 15 -in VCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	15.04	726.55	720	749	22.45	29	44.0	2127.3	90	77	3048	2659	0	1.21	1.18
25	14.55	726.55	720	749	22.45	29	43.6	2103.9	90	77	3048	2630	0	1.22	1.19
50	14.08	726.55	720	749	22.45	29	43.1	2081.2	90	77	3048	2601	0	1.24	1.21
100	13.19	726.55	720	749	22.45	29	42.2	2037.9	90	77	3048	2547	0	1.26	1.23
125	12.76	726.55	720	749	22.45	29	41.8	2017.4	90	77	3048	2522	0	1.27	1.24
150	12.35	726.55	720	749	22.45	29	41.3	1997.4	90	77	3048	2497	0	1.29	1.26
200	11.56	726.55	720	749	22.45	29	40.6	1959.5	90	77	3048	2449	0	1.31	1.28
300	10.14	726.55	720	749	22.45	29	39.1	1890.8	90	77	3048	2363	0	1.36	1.33
400	8.89	726.55	720	749	22.45	29	37.9	1830.5	90	77	3048	2288	0	1.40	1.37
500	7.80	726.55	720	749	22.45	29	36.8	1777.6	90	77	3048	2222	0	1.45	1.41

Ws Calc 15 -in VCP 1lf
90
Wc Calc
77

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

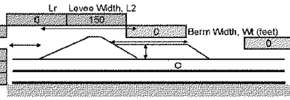
	Ground Elev 749		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 90		Wc = 77 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	749	0		90	77			
22.45	726.55	3048	2659	90	77	1.21	1.18	-212.44
23.45	725.55	3191	2778	90	77	1.21	1.18	
24.45	724.55	3335	2896	90	77	1.21	1.18	
25.45	723.55	3479	3014	90	77	1.21	1.18	
26.45	722.55	3623	3133	90	77	1.21	1.18	
27.45	721.55	3766	3251	90	77	1.21	1.19	
28.45	720.55	3910	3370	90	77	1.21	1.19	
29.45	719.55	4054	3488	90	77	1.21	1.19	
30.45	718.55	4198	3607	90	77	1.21	1.19	

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 90 lb/ft (15-inch Diameter, Standard Bell & Spigot VCP Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((15/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot



Conclusions
For levee stretch: 77+00 to 97+00
For pipe diam (in): 15
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

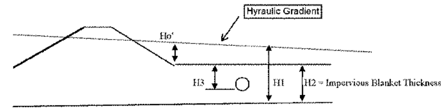
Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L6																							feet	
Stationing Range																								
77+00	400	400	29	29	29	50	18	0.84	0	0	1.63	0	150	762	912	912	15.04	15.04	15.04	0.52	762	762	0	0.000
97+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								



Levee Elev.	767
Ground Elev.	749
Blanket Base Elev.	720
Bedrock Elev.	670

18.45	Pipe Dept (ft)
732.55	Pipe Elev.
18	Pipe Diameter (in)
ML-CL	Soil Type
115	Soil Unit Weight, γsoil (pcf)
16.45	Pipe Depth (Check)

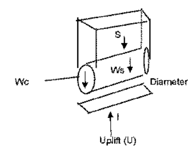
18" VCP entering MH 11



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 18 -in VCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	15.04	732.55	720	749	16.45	29	44.0	1558.8	141	110	2579	2338	0	1.21	1.16
25	14.55	732.55	720	749	16.45	29	43.6	1541.6	141	110	2579	2312	0	1.22	1.18
50	14.08	732.55	720	749	16.45	29	43.1	1524.9	141	110	2579	2287	0	1.24	1.19
100	13.19	732.55	720	749	16.45	29	42.2	1493.3	141	110	2579	2240	0	1.26	1.21
125	12.76	732.55	720	749	16.45	29	41.8	1478.2	141	110	2579	2217	0	1.28	1.23
150	12.35	732.55	720	749	16.45	29	41.3	1463.6	141	110	2579	2195	0	1.29	1.24
200	11.56	732.55	720	749	16.45	29	40.6	1435.8	141	110	2579	2154	0	1.31	1.26
300	10.14	732.55	720	749	16.45	29	39.1	1385.5	141	110	2579	2078	0	1.36	1.31
400	8.89	732.55	720	749	16.45	29	37.9	1341.3	141	110	2579	2012	0	1.41	1.35
500	7.80	732.55	720	749	16.45	29	36.8	1302.5	141	110	2579	1954	0	1.45	1.39

Ws Calc 18 -in VCP 1lf
141
Wc Calc
110

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 141 lb/ft (18-inch Diameter, Standard Bell & Spigot VCP Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1415*((18/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot



U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 749		Distance from toe (ft) 0					
		Ws = 141		Wc = 110 Full		Uplift Safety Extreme Case (1.1 REQ'D)		
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	749	0		141	110			
16.45	732.55	2579	2338	141	110	1.21	1.16	-147.91
17.45	731.55	2751	2480	141	110	1.21	1.17	
18.45	730.55	2924	2622	141	110	1.21	1.17	
19.45	729.55	3096	2765	141	110	1.21	1.17	
20.45	728.55	3269	2907	141	110	1.21	1.17	
21.45	727.55	3441	3049	141	110	1.21	1.17	
22.45	726.55	3614	3191	141	110	1.21	1.18	
23.45	725.55	3786	3333	141	110	1.21	1.18	
24.45	724.55	3959	3475	141	110	1.21	1.18	

Conclusions
For levee stretch: 77+00 to 97+00
For pipe diam (in): 18
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L6																							feet	
Stationing Range																								
77+00	400	400	29	29	29	50	18	0.84	0	0	1.63	0	150	762	912	912	15.04	15.04	15.04	0.52	762	762	0	0.000
97+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 0 — Levee Width, L2 150

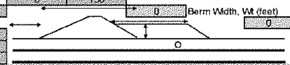
Levee Elev. 767 — Berm Width, Wt (feet) 0

Berm Height, t (feet) 0

Ground Elev. 749 — Pipe Dept (ft) 8

Blanket Base Elev. 720 — Pipe Elev. 743

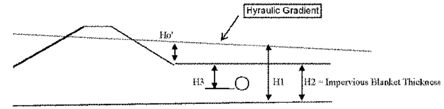
Bedrock Elev. 670 — Pipe Diameter (in) 24



Soil Type ML-CL

Soil Unit Weight, γsoil (pcf) 115

Pipe Depth (Check) 6



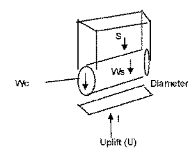
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 24 -in CIP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	15.04	743	720	749	6	29	44.0	568.5	264	196	920	1137	0	1.21	1.04
25	14.55	743	720	749	6	29	43.6	562.3	264	196	920	1125	0	1.23	1.05
50	14.08	743	720	749	6	29	43.1	556.2	264	196	920	1112	0	1.24	1.06
100	13.19	743	720	749	6	29	42.2	544.7	264	196	920	1089	0	1.27	1.09
125	12.76	743	720	749	6	29	41.8	539.2	264	196	920	1078	0	1.28	1.10
150	12.35	743	720	749	6	29	41.3	533.8	264	196	920	1068	0	1.29	1.11
200	11.56	743	720	749	6	29	40.6	523.7	264	196	920	1047	0	1.32	1.13
300	10.14	743	720	749	6	29	39.1	505.3	264	196	920	1011	0	1.37	1.17
400	8.89	743	720	749	6	29	37.9	489.2	264	196	920	978	0	1.41	1.21
500	7.80	743	720	749	6	29	36.8	475.1	264	196	920	950	0	1.45	1.25

Ws Calc
24 -in CIP 1lf
264
Wc Calc
196

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 264 lb/ft (30-inch Diameter, Class B, based on Concrete Pipe Design Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((18/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 749		Distance from toe (ft) 0		Uplift Safety Extreme Case (1.1 REQ'D)		
	Ws = 264		Wc = 196 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	749	0		264	196			
6	743	920	1137	264	196	1.21	1.04	66.80
7	742	1150	1327	264	196	1.21	1.07	
8	741	1380	1516	264	196	1.21	1.08	
9	740	1610	1706	264	196	1.21	1.10	
10	739	1840	1895	264	196	1.21	1.11	
11	738	2070	2085	264	196	1.21	1.12	
12	737	2300	2274	264	196	1.21	1.13	
13	736	2530	2464	264	196	1.21	1.13	
14	735	2760	2653	264	196	1.21	1.14	



U = P3 * diameter * l
Using l = 1

Conclusions
For levee stretch: 77+00 to 97+00
For pipe diam (in): 24
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L6																							feet	
Stationing Range																								
77+00	400	400	29	29	29	50	18	0.84	0	0	1.63	0	150	762	912	912	15.04	15.04	15.04	0.52	762	762	0	0.000
97+00																								
500-yr + 3ft																								

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.

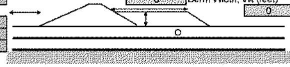
Lr 0 | Levee Width, L2 150

Levee Elev. 767 | Berm Width, Wt (feet) 0 | Berm Height, t (feet) 0

Ground Elev. 749

Blanket Base Elev. 720

Bedrock Elev. 670



15 Pipe Dept (ft)

734 Pipe Elev.

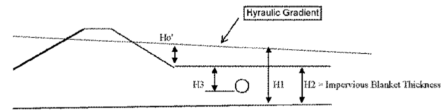
24 Pipe Diameter (in)

ML-CL Soil Type

115 Soil Unit Weight, γsoil (pcf)

15 Pipe Depth (Check)

24" VCP entering MH 10



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 24 -in VCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	15.04	734	720	749	15	29	44.0	1421.4	230	196	2990	2843	0	1.20	1.13
25	14.55	734	720	749	15	29	43.6	1405.7	230	196	2990	2811	0	1.22	1.15
50	14.08	734	720	749	15	29	43.1	1390.5	230	196	2990	2781	0	1.23	1.16
100	13.19	734	720	749	15	29	42.2	1361.6	230	196	2990	2723	0	1.25	1.18
125	12.76	734	720	749	15	29	41.8	1347.9	230	196	2990	2696	0	1.27	1.19
150	12.35	734	720	749	15	29	41.3	1334.6	230	196	2990	2669	0	1.28	1.21
200	11.56	734	720	749	15	29	40.6	1309.3	230	196	2990	2619	0	1.30	1.23
300	10.14	734	720	749	15	29	39.1	1263.3	230	196	2990	2527	0	1.35	1.27
400	8.89	734	720	749	15	29	37.9	1223.1	230	196	2990	2446	0	1.40	1.32
500	7.80	734	720	749	15	29	36.8	1187.7	230	196	2990	2375	0	1.44	1.36

Ws Calc 24 -in VCP 1lf
230
Wc Calc 196

See Sample Calculations for list of abbreviations and sample calculations

Ws = weight of structure per foot of length 230 lb/ft (24-inch Diameter, Standard Bell & Spigot VCP Manual)

Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((18/12)/2)^2 *1*62.4

S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil

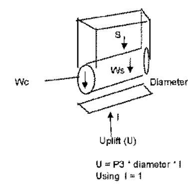
P3 = H3*(H1/H2)*γwater

U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3

Wg = weight of surcharge water above top surface of structure control by gravity flow

SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Ca = Weight of concrete anchor needed to prevent uplift per linear foot



Use this table to determine the distance from toe at which the Safety Factor is me

Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 749	Distance from toe (ft) 0				Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 230		Wc = 196 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	749	0		230	196			
15	734	2990	2843	230	196	1.20	1.13	-92.99
16	733	3220	3032	230	196	1.20	1.14	
17	732	3450	3222	230	196	1.20	1.14	
18	731	3680	3411	230	196	1.20	1.15	
19	730	3910	3601	230	196	1.20	1.15	
20	729	4140	3790	230	196	1.20	1.15	
21	728	4370	3980	230	196	1.21	1.16	
22	727	4600	4169	230	196	1.21	1.16	
23	726	4830	4359	230	196	1.21	1.16	

Conclusions

For levee stretch: 77+00 to 97+00

For pipe diam (in): 24

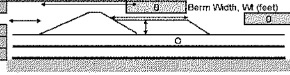
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee

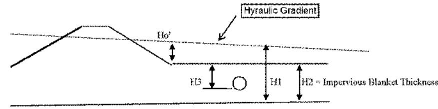
There is uplift concern at a depth of 8.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L6																							feet	
Stationing Range																								
77+00	400	400	29	29	29	50	18	0.84	0	0	1.63	0	150	762	912	912	15.04	15.04	15.04	0.52	762	762	0	0.000
97+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr Levee Width, L2

Lr	0
Levee Width, L2	150
Levee Elev.	767
Berm Width, Wt (feet)	0
Berm Height, t (feet)	0
Ground Elev.	749
Blanket Base Elev.	720
Bedrock Elev.	670
Pipe Dept (ft)	6
Pipe Elev.	743
Pipe Diameter (in)	30
Soil Type	ML-CL
Soil Unit Weight, γsoil (pcf)	115
Pipe Depth (Check)	6





d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 30 -in CIP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	15.04	743	720	749	6	29	44.0	568.5	384	306	1006	1421	0	1.19	0.98
25	14.55	743	720	749	6	29	43.6	562.3	384	306	1006	1406	0	1.21	0.99
50	14.08	743	720	749	6	29	43.1	556.2	384	306	1006	1391	0	1.22	1.00
100	13.19	743	720	749	6	29	42.2	544.7	384	306	1006	1362	0	1.25	1.02
125	12.76	743	720	749	6	29	41.8	539.2	384	306	1006	1348	0	1.26	1.03
150	12.35	743	720	749	6	29	41.3	533.8	384	306	1006	1335	0	1.27	1.04
200	11.56	743	720	749	6	29	40.6	523.7	384	306	1006	1309	0	1.30	1.06
300	10.14	743	720	749	6	29	39.1	505.3	384	306	1006	1263	0	1.34	1.10
400	8.89	743	720	749	6	29	37.9	489.2	384	306	1006	1223	0	1.39	1.14
500	7.80	743	720	749	6	29	36.8	475.1	384	306	1006	1188	0	1.43	1.17

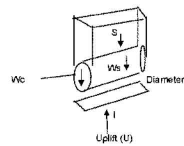
Ws Calc 30 -in CIP 1lf
384
Wc Calc 306

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 384 lb/ft (30-inch Diameter, Class B, based on Concrete Pipe Design Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((18/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 749	Distance from toe (ft) 140				Uplift Safety Extreme Case (1.1 REQ'D)		
	Ws = 384		Wc = 306 Full					
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	749	0		384	306			
6	743	1006	1340	384	306	1.27	1.04	83.60
7	742	1294	1563	384	306	1.27	1.07	
8	741	1581	1786	384	306	1.27	1.10	
9	740	1869	2010	384	306	1.27	1.12	
10	739	2156	2233	384	306	1.27	1.14	
11	738	2444	2456	384	306	1.28	1.15	
12	737	2731	2680	384	306	1.28	1.16	
13	736	3019	2903	384	306	1.28	1.17	
14	735	3306	3126	384	306	1.28	1.18	

Conclusions
For levee stretch: 77+00 to 97+00
For pipe diam (in): 30
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe



U = P3 * diameter * l
Using l = 1

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lf	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L6																							feet	
Stationing Range																								
77+00	400	400	29	29	29	50	18	0.84	0	0	1.63	0	150	762	912	912	15.04	15.04	15.04	0.52	762	762	0	0.000
97+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 0 — Levee Width, L2 150

Levee Elev. 767 — 0 Berm Width, Wt (feet)

0 Berm Height, t (feet)

Ground Elev. 749 — 15 Pipe Dept (ft)

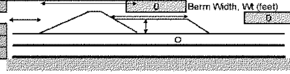
Blanket Base Elev. 720 — 734 Pipe Elev.

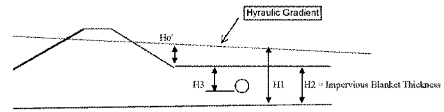
Bedrock Elev. 670 — 30 Pipe Diameter (in)

ML-CL Soil Type

115 Soil Unit Weight, γsoil (pcf)

15 Pipe Depth (Check)



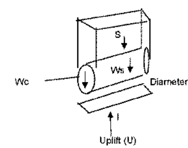


d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 30 -in VCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	15.04	734	720	749	15	29	44.0	1421.4	333	306	3594	3553	0	1.19	1.11
25	14.55	734	720	749	15	29	43.6	1405.7	333	306	3594	3514	0	1.20	1.12
50	14.08	734	720	749	15	29	43.1	1390.5	333	306	3594	3476	0	1.22	1.13
100	13.19	734	720	749	15	29	42.2	1361.6	333	306	3594	3404	0	1.24	1.15
125	12.76	734	720	749	15	29	41.8	1347.9	333	306	3594	3370	0	1.26	1.17
150	12.35	734	720	749	15	29	41.3	1334.6	333	306	3594	3336	0	1.27	1.18
200	11.56	734	720	749	15	29	40.6	1309.3	333	306	3594	3273	0	1.29	1.20
300	10.14	734	720	749	15	29	39.1	1263.3	333	306	3594	3158	0	1.34	1.24
400	8.89	734	720	749	15	29	37.9	1223.1	333	306	3594	3058	0	1.38	1.28
500	7.80	734	720	749	15	29	36.8	1187.7	333	306	3594	2969	0	1.43	1.32

Ws Calc
30 -in VCP 1lf
333

Wc Calc
306

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 333 lb/ft (30-inch Diameter, Standard Bell & Spigot VCP Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((30/12)/2)^2 * 1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot



U = P3 * diameter * l
Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 749	Distance from toe (ft) 0				Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 333		Wc = 306 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	749	0		333	306			
15	734	3594	3553	333	306	1.19	1.11	-17.99
16	733	3881	3790	333	306	1.19	1.11	
17	732	4169	4027	333	306	1.19	1.12	
18	731	4456	4264	333	306	1.19	1.12	
19	730	4744	4501	333	306	1.20	1.13	
20	729	5031	4738	333	306	1.20	1.13	
21	728	5319	4975	333	306	1.20	1.14	
22	727	5606	5212	333	306	1.20	1.14	
23	726	5894	5449	333	306	1.20	1.14	

Conclusions
For levee stretch: 77+00 to 97+00
For pipe diam (in): 30
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

CID-KS Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L6																							feet	
Stationing Range																								
77+00	400	400	29	29	29	50	16	0.84	0	0	1.63	0	150	762	912	912	15.04	15.04	15.04	0.52	762	762	0	0.000

97+00

500-yr + 3ft

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.

Lr 0 | Levee Width, L2 150

Levee Elev. 767 | Berm Width, Wt (feet) 0 | Berm Height, t (feet) 0

Ground Elev. 749 | Pipe Dept (ft) 3

Blanket Base Elev. 720 | Pipe Elev. 746

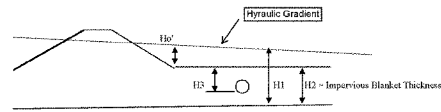
Bedrock Elev. 670 | Pipe Diameter (in) 8

Soil Type ML-CL

Soil Unit Weight, γsoil (pcf) 115

Pipe Depth (Check) 3

All elevations are based on NGVD of 1929.



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 8 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	15.04	746	720	749	3	29	44.0	284.3	29	22	179	190	0	1.21	1.09
25	14.55	746	720	749	3	29	43.6	281.1	29	22	179	187	0	1.22	1.11
50	14.08	746	720	749	3	29	43.1	278.1	29	22	179	185	0	1.24	1.12
100	13.19	746	720	749	3	29	42.2	272.3	29	22	179	182	0	1.26	1.14
125	12.76	746	720	749	3	29	41.8	269.6	29	22	179	180	0	1.28	1.15
150	12.35	746	720	749	3	29	41.3	266.9	29	22	179	178	0	1.29	1.17
200	11.56	746	720	749	3	29	40.6	261.9	29	22	179	175	0	1.31	1.19
300	10.14	746	720	749	3	29	39.1	252.7	29	22	179	168	0	1.36	1.23
400	8.89	746	720	749	3	29	37.9	244.6	29	22	179	163	0	1.41	1.27
500	7.80	746	720	749	3	29	36.8	237.5	29	22	179	158	0	1.45	1.31

Ws Calc 8 -in steel 1lf
28.55
Wc Calc 22

See Sample Calculations for list of abbreviations and sample calculations

Ws = weight of structure per foot of length = based on 8-inch Nom Diam Steel Pipe, OD 8.625" wall thickness .322"

Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((4/12)^2) *1*62.4

S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil

P3 = H3*(H1/H2)*γwater

U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3

Wg = weight of surcharge water above top surface of structure control by gravity flow

SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Ca = Weight of concrete anchor needed to prevent uplift per linear foot

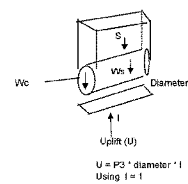
Use this table to determine the distance from toe at which the Safety Factor is me

Enter the Distance from toe and change the value until the SFf is equal to that required

Ground Elev 749 | Distance from toe (ft) 0

Ws = 29 | Wc = 22 Full | Uplift Safety Extreme Case (1.1 REQ'D)

Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	749	0		29	22			
3	746	179	190	29	22	1.21	1.09	1.02
4	745	256	253	29	22	1.21	1.12	
5	744	332	316	29	22	1.21	1.14	
6	743	409	379	29	22	1.21	1.15	
7	742	486	442	29	22	1.21	1.16	
8	741	562	505	29	22	1.21	1.17	
9	740	639	569	29	22	1.21	1.17	
10	739	716	632	29	22	1.21	1.18	
11	738	792	695	29	22	1.21	1.18	



Conclusions

For levee stretch: 77+00 to 97+00

For pipe diam (in): 8

There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee

There is uplift concern at a depth of 8.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L7																							feet	
Stationing Range																								
97+00	300	300	35	35	35	50	12.5	0.84	0	0	2.69	0	100	725	825	825	10.98	10.98	10.98	0.31	725	725	0	0.000
107+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr 0 | Levee Width, L2 100

Levee Elev. 767.5

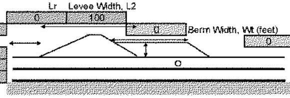
Berm Width, Wt (feet) 0

Berm Height, t (feet) 0

Ground Elev. 755

Blanket Base Elev. 720

Bedrock Elev. 670



15 Pipe Dept (ft)

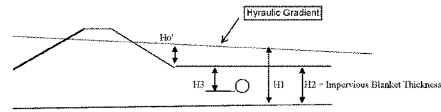
740 Pipe Elev.

12 Pipe Diameter (in)

ML Soil Type

115 Soil Unit Weight, γsoil (pcf)

15 Pipe Depth (Check)



d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in VCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.98	740	720	755	15	35	46.0	1229.7	54	49	1610	1230	0	1.39	1.35
25	10.61	740	720	755	15	35	45.6	1219.8	54	49	1610	1220	0	1.40	1.36
50	10.25	740	720	755	15	35	45.3	1210.2	54	49	1610	1210	0	1.42	1.38
100	9.57	740	720	755	15	35	44.6	1191.9	54	49	1610	1192	0	1.44	1.40
125	9.24	740	720	755	15	35	44.2	1183.2	54	49	1610	1183	0	1.45	1.41
150	8.93	740	720	755	15	35	43.9	1174.8	54	49	1610	1175	0	1.46	1.42
200	8.33	740	720	755	15	35	43.3	1158.9	54	49	1610	1159	0	1.48	1.44
300	7.26	740	720	755	15	35	42.3	1130.2	54	49	1610	1130	0	1.52	1.47
400	6.32	740	720	755	15	35	41.3	1105.1	54	49	1610	1105	0	1.55	1.51
500	5.51	740	720	755	15	35	40.5	1083.3	54	49	1610	1083	0	1.58	1.54

Ws Calc 12 -in VCP 1lf
54
Wc Calc 49

See Sample Calculations for list of abbreviations and sample calculations

Ws = weight of structure per foot of length 54 lb/ft (36-inch Diameter, Standard Bell & Spigot VCP Manual)

Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1415*(18/12)^2) *1*62.4

S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil

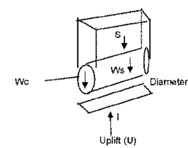
P3 = H3*(H1/H2)*γwater

U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3

Wg = weight of surcharge water above top surface of structure control by gravity flow

SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Ca = Weight of concrete anchor needed to prevent uplift per linear foot



U = P3 * diameter * l

Using l = 1

Use this table to determine the distance from toe at which the Safety Factor is me

Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 755		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 54		Wc = 49 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	755	0		54	49			
15	740	1610	1230	54	49	1.39	1.35	-311.28
16	739	1725	1312	54	49	1.39	1.36	
17	738	1840	1394	54	49	1.39	1.36	
18	737	1955	1476	54	49	1.39	1.36	
19	736	2070	1558	54	49	1.40	1.36	
20	735	2185	1640	54	49	1.40	1.37	
21	734	2300	1722	54	49	1.40	1.37	
22	733	2415	1804	54	49	1.40	1.37	
23	732	2530	1886	54	49	1.40	1.37	

Conclusions

For levee stretch: 97+00 to 107+00

For pipe diam (in): 12

There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee

There is uplift concern at a depth of 8.33 feet and less at the landward toe

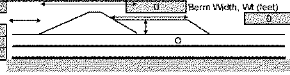
Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L7																							feet	
Stationing Range																								
97+00	300	300	35	35	35	50	12.5	0.84	0	0	2.69	0	100	725	825	825	10.98	10.98	10.98	0.31	725	725	0	0.000

107+00
500-yr + 3ft

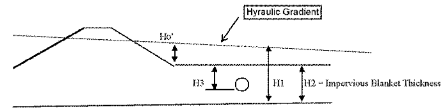
Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.

Lr Levee Width, L2
0 100

Levee Elev. 787.5 — 0 Berm Width, Wt (feet)
0 Berm Height, t (feet)



Ground Elev.	755	25	Pipe Dept (ft)
Blanket Base Elev.	720	730	Pipe Elev.
Bedrock Elev.	670	15	Pipe Diameter (in)
		ML	Soil Type
		115	Soil Unit Weight, γsoil (pcf)
		25	Pipe Depth (Check)

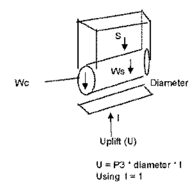


d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 15 -in VCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.98	730	720	755	25	35	46.0	2049.6	90	77	3414	2562	0	1.40	1.37
25	10.61	730	720	755	25	35	45.6	2033.0	90	77	3414	2541	0	1.41	1.38
50	10.25	730	720	755	25	35	45.3	2016.9	90	77	3414	2521	0	1.42	1.39
100	9.57	730	720	755	25	35	44.6	1986.5	90	77	3414	2483	0	1.44	1.41
125	9.24	730	720	755	25	35	44.2	1972.0	90	77	3414	2465	0	1.45	1.42
150	8.93	730	720	755	25	35	43.9	1958.0	90	77	3414	2448	0	1.46	1.43
200	8.33	730	720	755	25	35	43.3	1931.5	90	77	3414	2414	0	1.48	1.45
300	7.26	730	720	755	25	35	42.3	1883.6	90	77	3414	2354	0	1.52	1.49
400	6.32	730	720	755	25	35	41.3	1841.9	90	77	3414	2302	0	1.56	1.52
500	5.51	730	720	755	25	35	40.5	1805.5	90	77	3414	2257	0	1.59	1.55

Ws Calc
15 -in VCP 1lf
90

Wc Calc
77

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 90 lb/ft (15-inch Diameter, Standard Bell & Spigot VCP Manual)
Wc = weight of water contained in the structure = pi * r^2 * l = 3.1416*((18/12)^2) * 1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot



Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 755		Distance from toe (ft) 0			Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 90		Wc = 77 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	755	0		90	77			
25	730	3414	2562	90	77	1.40	1.37	-685.90
26	729	3558	2664	90	77	1.40	1.37	
27	728	3702	2767	90	77	1.40	1.37	
28	727	3845	2869	90	77	1.40	1.37	
29	726	3989	2972	90	77	1.40	1.37	
30	725	4133	3074	90	77	1.40	1.37	
31	724	4277	3177	90	77	1.40	1.37	
32	723	4420	3279	90	77	1.40	1.38	
33	722	4564	3382	90	77	1.40	1.38	

Conclusions
For levee stretch: 97+00 to 107+00
For pipe diam (in): 15
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

Armourdale Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-e	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L7																							feet	
Stationing Range																								
97+00	300	300	35	35	35	50	12.5	0.84	0	0	2.69	0	100	725	825	825	10.98	10.98	10.98	0.31	725	725	0	0.000
107+00	Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.																							
500-yr + 3ft																								

Lr Levee Width, L2: 0 | 100

Levee Elev. 767.5

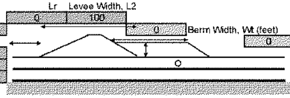
Berm Width, Wt (feet): 0

Berm Height, t (feet): 0

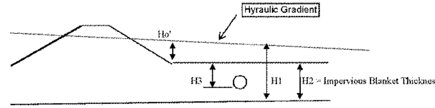
Ground Elev. 755

Blanket Base Elev. 720

Bedrock Elev. 670



25	Pipe Depth (ft)
730	Pipe Elev.
24	Pipe Diameter (in)
ML	Soil Type
115	Soil Unit Weight, γsoil (pcf)
25	Pipe Depth (Check)



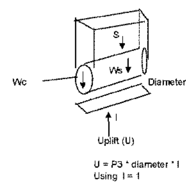
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 24 -in VCP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.98	730	720	755	25	35	46.0	2049.6	230	196	5290	4099	0	1.39	1.35
25	10.61	730	720	755	25	35	45.6	2033.0	230	196	5290	4066	0	1.41	1.36
50	10.25	730	720	755	25	35	45.3	2016.9	230	196	5290	4034	0	1.42	1.37
100	9.57	730	720	755	25	35	44.6	1986.5	230	196	5290	3973	0	1.44	1.39
125	9.24	730	720	755	25	35	44.2	1972.0	230	196	5290	3944	0	1.45	1.40
150	8.93	730	720	755	25	35	43.9	1958.0	230	196	5290	3916	0	1.46	1.41
200	8.33	730	720	755	25	35	43.3	1931.5	230	196	5290	3863	0	1.48	1.43
300	7.26	730	720	755	25	35	42.3	1883.6	230	196	5290	3767	0	1.52	1.47
400	6.32	730	720	755	25	35	41.3	1841.9	230	196	5290	3684	0	1.55	1.50
500	5.51	730	720	755	25	35	40.5	1805.5	230	196	5290	3611	0	1.58	1.53

Ws Calc 24 -in VCP lIf
230
Wc Calc 196

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 230 lb/ft (24-inch Diameter, Standard Ball & Spigot VCP Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((18/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
Ground Elev 755		Ws = 230	Distance from toe (ft) 0	Wc = 196 Full		Uplift Safety Extreme Case (1.1 REQ'D)		
	755	0		230	196			
25	730	5290	4099	230	196	1.39	1.35	-1010.93
26	729	5520	4263	230	196	1.39	1.35	
27	728	5750	4427	230	196	1.40	1.35	
28	727	5980	4591	230	196	1.40	1.35	
29	726	6210	4755	230	196	1.40	1.35	
30	725	6440	4919	230	196	1.40	1.36	
31	724	6670	5083	230	196	1.40	1.36	
32	723	6900	5247	230	196	1.40	1.36	
33	722	7130	5411	230	196	1.40	1.36	



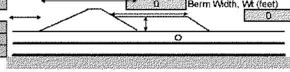
Conclusions
For levee stretch: 97+00 to 107+00
For pipe diam (in): 24
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

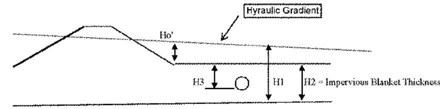
CID-KS Area	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
L10																							feet	
Stationing Range																								
127+00	500	500	27	27	27	58	16	0.84	0	0	1.54	0	70	885	955	955	11.50	14.83	14.83	0.55	885	885	0	0.000
168+00																								
500-yr + 3ft																								

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineers and is used to develop the Hydraulic Gradient at various distances from the toe.

Lr Levee Width, L2

Lr	0
Levee Width, L2	70
Levee Elev.	771
Berm Width, Wt (feet)	0
Berm Height, t (feet)	0
Ground Elev.	755
Blanket Base Elev.	728
Bedrock Elev.	670
Pipe Dept (ft)	6
Pipe Elev.	749
Pipe Diameter (in)	24
Soil Type	CL
Soil Unit Weight, γsoil (pcf)	115
Pipe Depth (Check)	6





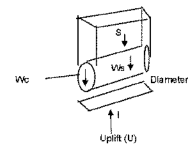
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 24 -in CIP	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	11.50	749	728	755	6	27	38.5	533.9	264	196	920	1068	0	1.29	1.11
25	11.50	749	728	755	6	27	38.5	533.9	264	196	920	1068	0	1.29	1.11
50	11.50	749	728	755	6	27	38.5	533.9	264	196	920	1068	0	1.29	1.11
100	11.50	749	728	755	6	27	38.5	533.9	264	196	920	1068	0	1.29	1.11
125	11.50	749	728	755	6	27	38.5	533.9	264	196	920	1068	0	1.29	1.11
150	11.50	749	728	755	6	27	38.5	533.9	264	196	920	1068	0	1.29	1.11
200	11.50	749	728	755	6	27	38.5	533.9	264	196	920	1068	0	1.29	1.11
300	11.50	749	728	755	6	27	38.5	533.9	264	196	920	1068	0	1.29	1.11
400	11.50	749	728	755	6	27	38.5	533.9	264	196	920	1068	0	1.29	1.11
500	11.50	749	728	755	6	27	38.5	533.9	264	196	920	1068	0	1.29	1.11

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 755		Distance from toe (ft) 0		Uplift Safety Extreme Case (1.1 REQ'D)		
		Ws = 264	Wc = 196 Full				
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty	Ca Lbs/ft
	755	0		264	196			
6	749	920	1068	264	196	1.29	1.11	-9.49
7	748	1150	1246	264	196	1.29	1.14	
8	747	1380	1424	264	196	1.29	1.15	
9	746	1610	1602	264	196	1.29	1.17	
10	745	1840	1780	264	196	1.29	1.18	
11	744	2070	1958	264	196	1.29	1.19	
12	743	2300	2135	264	196	1.29	1.20	
13	742	2530	2313	264	196	1.29	1.21	
14	741	2760	2491	264	196	1.29	1.21	

Ws Calc
24 -in CIP 1lf
264
Wc Calc
196

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length 264 lb/ft (30-inch Diameter, Class 8, based on Concrete Pipe Design Manual)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((18/12)^2) *1*62.4
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)
Ca = Weight of concrete anchor needed to prevent uplift per linear foot



Uplift (U)

U = P3 * diameter * l
Using l = 1

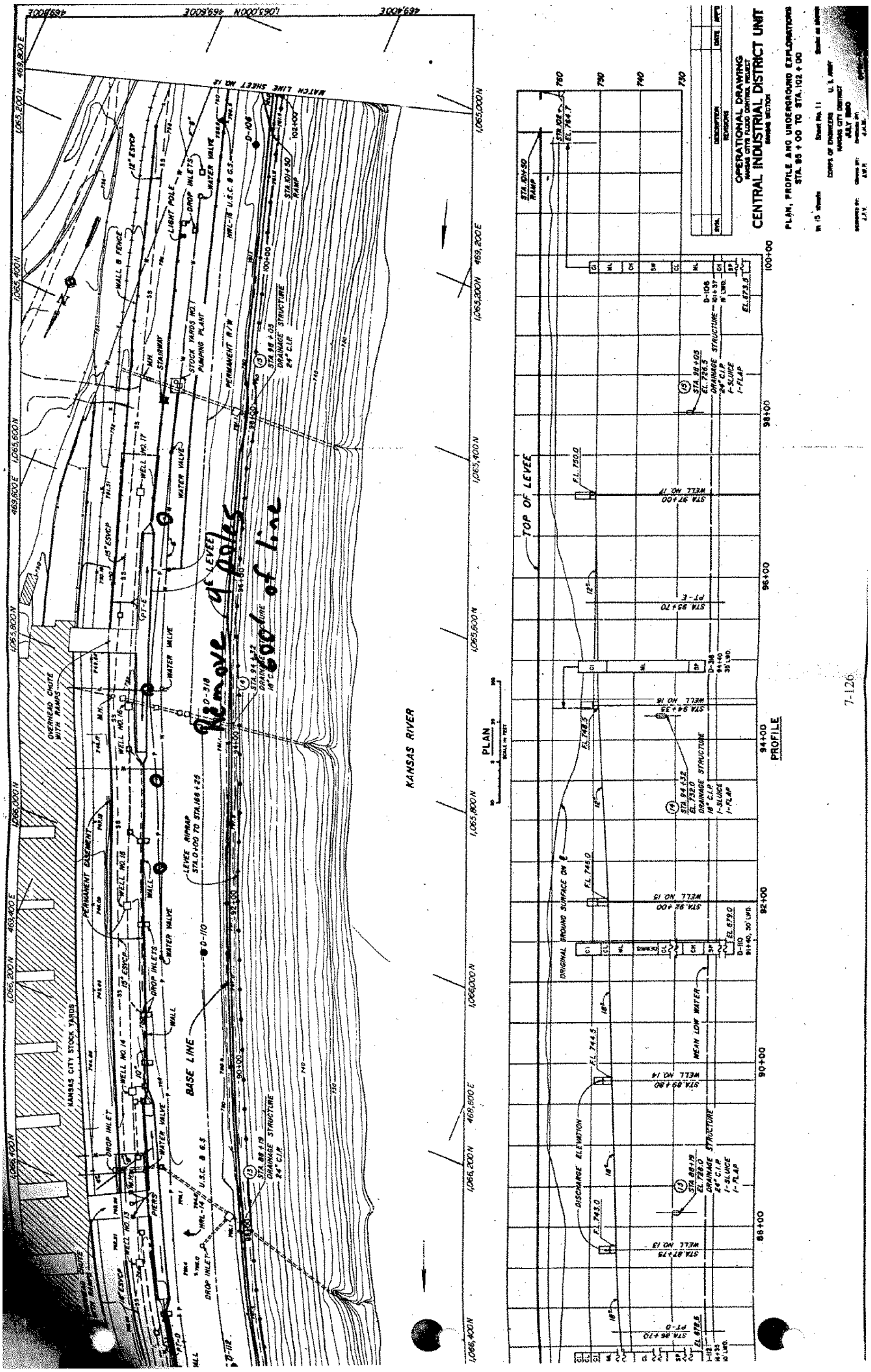
Conclusions
For levee stretch: 127+00 to 168+00
For pipe diam (in): 24
There is uplift concern at a depth of 40 inches (3.3 feet) and less within 500 feet landward of the levee
There is uplift concern at a depth of 8.33 feet and less at the landward toe

EXHIBIT A-7.9

Power Line Relocations Calculations

STA ~ 94+00

Gateway 2000 Pump Plant to Field pump house
Remove 4 power poles
Remove Approx 600 ft/strand assume 3 strands


KANSAS RIVER
PLAN
PROFILE
TOP OF LEVEE
BASE LINE
KANSAS CITY STOCK YARDS
OPERATIONAL DRAWING
CENTRAL INDUSTRIAL DISTRICT UNIT
Remove 4 poles
600' of line
7-126

Approx 57+25 to 73+80

Relocate 15 wooden power poles 1500' of wire
3 strands

Relocate approx 20' to toe of new levee.

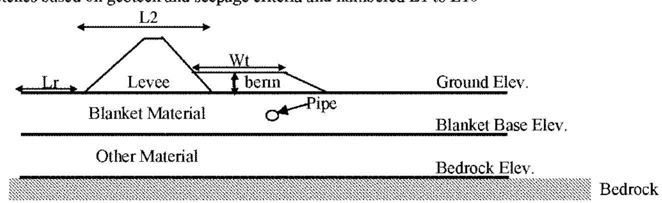
EXHIBIT A-7.10

CID-KS Utility Uplift Spreadsheet: Data Entry Worksheet

CID-KS Uplift: Data Entry Worksheet			
Kansas City Levees Phase 2			
Created By:	Cassidy Garden	Date:	2-Jun-08
		Date Modified:	2-Jun-08
Peer Reviewed By:	Hank Mildenberger	Date:	

How to Use this Spreadsheet

Insert parameters into cells highlighted in orange and the corresponding spreadsheets will update automatically.
Spreadsheets are linked, therefore spreadsheets must stay in their original directories to maintain links.
CID Unit was separated ten stretches based on geotech and seepage criteria and numbered L1 to L10



CID-KS Uplift: Data Entry Worksheet
Kansas City Levees Phase 2

Levee Type Number		L1	L2	L3	L4	L5	L6	L7	L8	L9	L10
Station Start		80+54.12 MO	28+00	38+00	63+00	74+75	77+00	97+00	107+00	116+00	127+00
Station End		28+00 KS	38+00	63+00	74+75	77+00	97+00	107+00	116+00	127+00	168+00
Levee Width, L2 (from geotech)		40	65	60	100	150	150	100	70	70	70
Riverside Blanket Width, Lr (from geotech)		0	0	0	0	0	0	0	0	0	0
Top of Levee Elev. (n500 + 3ft)		760.9	762.0	763.0	765.0	765.5	767.0	767.5	768.0	769.0	771.0
Berm Width for the w/ Berm option, Wt (ft) (from geotech)		NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
Berm Height for the w/Berm option, t (feet) (from geotech)		NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
Ground Elev. Landside		760	748	751	751	760	749	755	752	757	755
Blanket Bottom Elev. Landside (from Geotech)		726	721	720	720	722	720	720	720	721	728
Bedrock Elev. (from O&M manual Borings)		670	670	670	670	670	670	670	670	670	670
Pipe Depth ft. (enter on spreadsheet		Varies	Varies	Varies	Varies	Varies	Varies	Varies	Varies	Varies	Varies
Pipe Diameter in. (enter on spreadsheet)		Varies	Varies	Varies	Varies	Varies	Varies	Varies	Varies	Varies	Varies
Blanket Soil Type (from geotech)		ML	CH	ML-CL	ML-CL	CL	ML-CL	ML	ML	ML	CL
Blanket Soil Unit Weight, soil density (pcf)		115	115	115	115	115	115	115	115	115	115
Blanket Thickness Dbo ft (from geotech EC-GD) landside		34	27	31	31	38	29	35	32	36	27
Depth of Sands Df ft (from Geotech EC-GD)		56	51	50	50	52	50	50	50	51	58
Max Head or Levee Height ft (from geotech EC-GD)		0.9	14.0	12.0	14.0	5.5	18.0	12.5	16.0	12.0	16.0
Notes			Area Fill		Area Fill	Bridge Fill	Area Fill		Relief Wells		Relief Wells

Input Data	
Calculated	
NA	Not Applicable

CID-KS Uplift: Data Entry Worksheet
Kansas City Levees Phase 2

NOMENCLATURE for all Uplift Spreadsheets

Input

(Kf/Kb)R = riverside permeability
(Kf/Kb)L = landside permeability
DbL = landside blanket thickness
Dbr = riverside blanket thickness
Dbo = levee toe blanket thickness
Df = thickness of pervious foundation
Lr = length of riverside blanket
LL = length of landside blanket
H = max head or levee height
L2 = levee base width
t = berm height, ft
ground elevation = average elevation of landside ground

Output

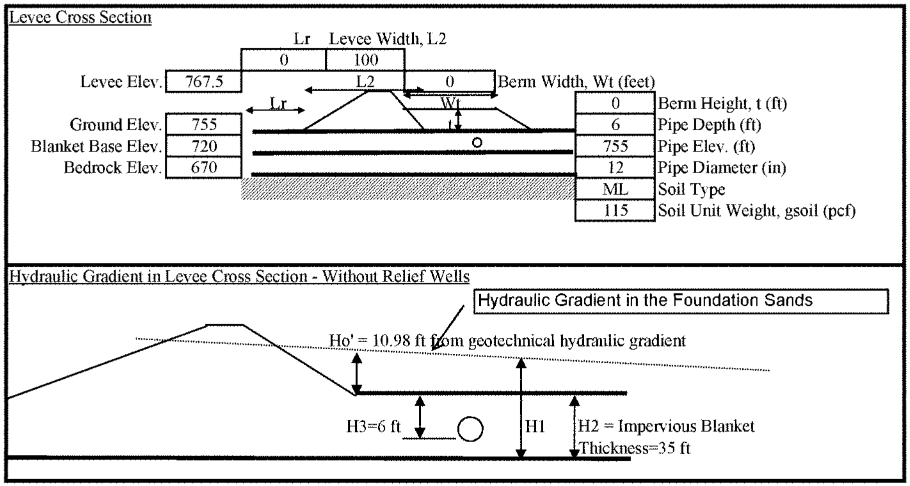
Cr = riverside effective length coefficient
CL = landside effective length coefficient
where C = [(Kf/Kb) * Df * Db]1/2
L1 = riverside effective length
where L1 = C * (e (2LR/C-1)) / (e(2LR/C+1))
Le = landside effective length
Lt = total effective length
ho = head above tailwater at levee toe
io = seepage gradient
ic = critical gradient = (γsat-γwater)/γwater

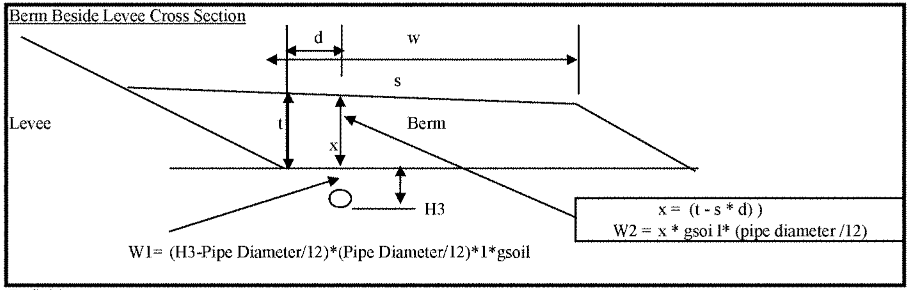
EXHIBIT A-7.11

Sample Calculation for Utility Uplift

Kansas City's Levees - Phase 2 - DRAFT
CID-KS - Utility Uplift Sample Calculation at Station 100+00 (Distance from toe - 0ft)

By: Cassidy Garden	Date Created:	14-Aug-07
	Date Modified:	7-Nov-08
Peer Reviewed By: Hank Mildenberger	Date Reviewed:	30-Nov-08





Definitions

H1 = Height of Hydraulic Gradient above base of Impervious Blanket - ft
H2 = Impervious Blanket Thickness - ft
H3 = Depth of Pipe Invert -ft
Ho' = Excess head above ground surface (initial Ho' is calculated at toe of levee by geotechnical engineer) - ft
Lr = Length of riverside blanket (determined by geotechnical engineer) - ft
L1 = Riverside effective length (calculated by geotechnical engineer) - ft
L2 = Levee Width (toe to toe) - ft
Wt = Berm Width - ft
t = Berm Height at toe of levee- ft
x = Berm Height at structure (pipe)- ft
s = slope of berm
Ws = weight of structure (pipe) per foot of length
18.97 lb per ft (12" Diameter Steel Pipe, .375" wall thickness, Manual of Steel Construction)
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*((3/12)^2) *1*62.4 = 12 lb/lf
S = surcharge loads = weight of saturated soils above structure = W1 + W2
W1 = Surcharge load above structure (not including berm)
W2 = Surcharge load of berm above structure
P1 = Pressure at the base of the impervious blanket at the location of the structure being investigated
P3 = Pressure at the base of the structure (pipe) being investigated
U = Uplift force on the project area of structure = Area of pipe * P3
Wg = weight of surcharge water above top surface of structure control by gravity flow = 0 for pipes
SFf = Flotation Safety Factor = (Ws+Wc+S)/(U-Wg)

<u>Sample Calculations</u>
Sample Calculations are done at the toe of the levee (distance from toe = 0 ft)
H1
H1 = Ho' + Ground Elev - Impervious Blanket Base Elev
H1 = 10.98 ft + 755 ft - 720 ft = 45.98 ft

P3
P3 = H3 x (H1/H2) x gwater
P3 = 6 ft x (45.98ft /35ft) x 62.4 pcf = 491.85 psf

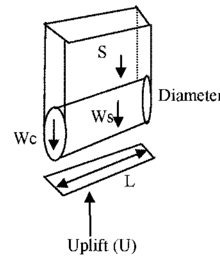
Wc
Wc = pi x r^2 x gwater
Wc = 3.1416 x ((6"/12)^2) x 62.4 pcf = 49 lb/lf

S
S = W1 + W2
W1 = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*gsoil
W1 = (6 ft - 12"/12) x (12"/12) x 115 pcf x 1 lf
W1 = 575 pounds
W2 = (t - s x d) (Pipe Diameter/12) x 1 lf x gsoil
W2 = (0 - (0.00 x 0))x (12/12 x 1 lf x 115 psf) = 0 pounds
S = W1 + W2 = 575 + 0 = 575 pounds

U
U = Area of pipe * P3 = (Pipe Diameter/12) x 1 ft x P3
U = 12"/12 x 491.85 psf x 1 ft
U = 491.85 lb

SFf (Pipe Full) (w/o berm)
SFf = Flotation Safety Factor = (Ws+Wc+S)/(U-Wg)
SFf = (49.56 + 49 + 575)/(491.85-0)
SFf = 1.37

SFf (Pipe Empty) (w/o berm)
SFf = (49.56 + 575)/(491.85-0)
SFf = 1.27



U = P3 * diameter * 1
Using L = 1

Exhibit A-7.12

CID-KS Utility Uplift Summary **Date: 14 July 2008**

Kansas City Levees Phase 2

Created By: Cassidy Garden Modified By:

Peer Reviewed By: Hank Mildenberger Modified: 14-Jul-08

Length of Utility (feet) and number of Manholes within Uplift Concern Area

	Station		Storm Sewer							Water							Gas	
Levee	Begin	End	12"	15"	18"	24"	30"	36"	MH	8"	12"	20"	24"	30"	48"	72"	16"	20"
L1	80+54.12	28+00 KS	No Up Lift Concerns							No Up Lift Concerns							N U L C	
L2	28+00	38+00	No Up Lift Concerns							No Up Lift Concerns							N U L C	
L3	38+00	63+00	No Up Lift Concerns								50						N U L C	
L4	63+00	74+75	No Up Lift Concerns							No Up Lift Concerns							N U L C	
L5	74+75	77+00	No Up Lift Concerns							No Up Lift Concerns							N U L C	
L6	77+00	97+00					650		5	No Up Lift Concerns							N U L C	
L7	97+00	107+00	No Up Lift Concerns							No Up Lift Concerns							N U L C	
L8	107+00	116+00	No Up Lift Concerns							No Up Lift Concerns							N U L C	
L9	116+00	127+00	No Up Lift Concerns							No Up Lift Concerns							N U L C	
L10	127+00	168+00	No Up Lift Concerns							No Up Lift Concerns							N U L C	
Total			0	0	0	0	650	0	5	0	50	0	0	0	0	0	0	0

MH Manhole
SS Sanitary Sewer
FM Forcemain
NULC No Up Lift Concerns

Unified Government of Wyandotte County AutoCAD maps were used to determine depth of Sanitary Sewer Lines when possible. If no depth was provided on AutoCAD maps, then a depth of 40" was assumed. This assumption is conservative and would show more cases of uplift concern then may be present.

Kansas Citys, Missouri and Kansas
Flood Risk Management Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Final Feasibility Report

Chapter A-8

GENERAL STRUCTURAL ANALYSIS

CHAPTER A-8
GENERAL STRUCTURAL ANALYSIS

A-8.1 STRUCTURAL ANALYSIS METHODOLOGY

A-8.1.1 Introduction

The structural features of the levee units included in this study consist of floodwalls, pump stations, closure structures for openings in levees and floodwalls, gatewells, reinforced box culverts, drainage structures, and retaining walls integral to the integrity of the levee system. The evaluation of each unit's structures includes the assessment of existing conditions and formulation of several project alternatives. These alternatives include raising the level unit to meet the elevation of a Nominal 500 year flood (N500+0), an N500 plus three additional feet (N500+3), and an N500 plus five additional feet (N500+5). The findings then provide input to the HEC-FDA economics model used to develop benefit to cost (B/C) ratios and an economic assessment.

For existing conditions, the features were analyzed at various water levels to establish reliability as required by the HEC-FDA program model. This work was based on visual observation, dated construction plans, historical data, and discussions with the Corps of Engineers and Levee District personnel (those familiar with and involved in the inspection, operation, and maintenance of the levee units), detailed engineering analysis, and engineering judgment.

Alternatives for creating flood control protection above the current level of protection included removal and replacement of floodwall with earthen levee, modification, and new construction. For modifications, similar work to that stated above for existing structures was required including assumptions and engineering judgment. Analysis of the N500+0, N500+3, and N500+5 focused on the N500+3 event and those findings were extrapolated to estimate requirements for the N500+0 and N500+5 where possible. The B/C ratios for future conditions were also used in the development of the economic curve.

The work contained herein is to provide feasibility analysis results only; it does not replace a deterministic design analysis, or answer the questions that only a deterministic design analysis can. The structures were analyzed without factors of safety and with consistent assumptions, in order to evaluate the relative risk and consequences for economic and risk-informed decision-making purposes. Risk and reliability studies do not replace deterministic analyses, nor do such studies confirm the satisfaction of any design criteria, past or present. They simply provide additional information for the decision-maker with respect to the possible performance of the structure for the loads under consideration. This provides a risk-informed decision with respect to project repairs or improvements.

A-8.1.2 Deterministic Design Criteria

A series of screening criteria are used to determine if a probabilistic analysis is necessary for a given structure. Summarized below are some general assumptions used to analyze structural components as well as the strength and stability criterion from the current design standards. If the analysis showed that the existing structural component meets the below criteria, it was assumed reliable and a 99.8% reliability was assigned. If the structural component did not meet the criteria, a reliability analysis was performed.

A-8.1.2.1 General Assumptions

The following lists major assumptions in the feasibility study. Other assumptions specific to a feature are noted in their respective levee system appendix chapters.

1. Some structural components were not analyzed. Only components judged to be critical based upon engineering experience were analyzed for feasibility.
2. The structural components were analyzed based on dimensions, quantities, and conditions represented by record drawings. Deviations from plans cannot be verified per scope and budget. This is a consistent assumption for relative risk and reliability assessment for Kansas City's levee feasibility studies.
3. Parts of the components being analyzed that were not evaluated with this analysis include, but may not be limited to, minimum rebar embedment lengths, structure capacity at rebar cutoff locations, axial tension in heels due to resisting pressures on the key, various cut-off wall efficiencies, and various soil resistances.
4. Secondary and incremental load effects are not considered for the feasibility analysis. Permanent deformation or damages from any less than extreme floods that occur prior to the extreme flood are expected to have been repaired to minimum USACE criteria.
5. Structural materials, such as reinforcing bar and concrete, are in good condition, without voids or other significant defect.
6. Soil is adequately compacted and fill type and strength parameters supplied by the Geotechnical Section is correct.
7. Cut-off walls are 50% efficient, and are of adequate strength and good condition. The upstream face of structural wedges was analyzed with a Line of Creep, reduction beginning at top of ground. This is the default method in CTWALL. Toe drains are considered inoperable.

A-8.1.2.2 Stability and Pile Capacity Requirements

Pile capacity requirements are based on EM 1110-2-2906 Design of Pile Foundations. Structural stability criterion can be seen in Table A-8.1. It is based upon EM 1110-2-2100 Stability Analysis of Concrete Structures (01 Dec 2005), with the exception of the extreme load condition. The Missouri River L-142 Design Criteria Issue Resolution Paper (2002) addressed a concern with the extreme load condition categories as specified in EM 1110-2-2100 and put forth more stringent guidelines for recommended extreme load condition stability criteria. That criterion is used herein.

TABLE A-8.1
Stability Criterion

Recommended Sliding Stability Factor of Safety		
Load Condition Category	Return Period	Factor of Safety
Usual	10 yrs	2
Unusual	300 yrs	1.5
Extreme	Top of Protection	1.3*

Recommended Rotational Stability Percent of Base in Compression		
Load Condition Category	Return Period	Percent of Base in Compression
Usual	10 yrs	100%
Unusual	300 yrs	75%
Extreme	Top of Protection	25% *

Recommended Maximum Allowable Bearing Capacity % Increase in Allowable Bearing Capacity		
Load Condition Category	Return Period	% Increase in Allowable Bearing Capacity
Usual	10 yrs	0%
Unusual	300 yrs	15%
Extreme	Top of Protection	50%

Recommended Flotation Stability Factor of Safety		
Load Condition Category	Return Period	Factor of Safety
Usual	10 yrs	1.3
Unusual	300 yrs	1.2
Extreme	Top of Protection	1.1

* Stability requirements increased from value in EM 1110-2-2100

A-8.1.2.3 Strength Requirements

Strength requirements are based on the Strength Design Method as outlined by EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures*. Dead and live load factor (LF) of 1.7 and a hydraulic factor (HF) of 1.3 (when applicable). The respective ACI Strength Reduction Factors are used with the corresponding Load Factors above (see ACI 350 Appendix C). The strength reduction factor for tension-controlled sections (ϕ) is 0.90; the strength reduction factor for shear and torsion is 0.85. Other strength reduction factors can be found in ACI 350-06, Appendix C.

A-8.1.3 Structural Reliability Methodology - Existing Structures Only

The following structural methodology was developed by the Kansas City District during the course of the Phase 1 – Kansas Citys Levees Feasibility Study. The subsequent criterion was accepted by representatives of the U.S. Army Corps of Engineers Headquarters in the Fall of 2005. The approved structural reliability methodology referred to below, can be found in the MFR "Kansas Citys Structural Summit held 01 Dec 05", Memorandum dated 13 Jan 06.

A-8.1.4 Deterministic Criteria - Existing Structures Only

Typical strength reduction factors and load factors were not used in the analysis of these structures. Load factors and reduced strengths are used in design, but are considered inappropriate for a probability of failure analysis. If an existing structure has a calculated factor of safety of less than 1.0 (Capacity/Demand), then it implies failure of that structure.

A high enough Factor of Safety (FS) in the strength analysis will provide 99.8% reliability because any variance in coefficients is too low to overcome the safety factor. There is a limit where the FS is still above 1, yet the probability of failure (POF) will begin to increase due to statistical possibilities presented by the coefficient of variance. To prevent unnecessary POF analyses, it is desirable to determine this FS threshold. Two reasons are given to set this FS threshold at 1.5. First, it is possible to calculate the maximum range of FS based upon the coefficients of variation used in the analysis. This is performed by likening the analysis to measurement and instrumentation. Coefficients of Variation (COV) are then treated like uncertainty of a measurement (FS) based upon the mean values. For a system with N^{th} order of uncertainty, a 95% confidence estimate of total uncertainty can be computed by the square root of the sum of the squares of each coefficient of variation. Considering the Coefficient of Variation for concrete compressive strength, steel yield strength, unit weight of soil, seepage pressures, and the angle of internal friction yields a probable maximum range in FS of +/- .28. Failure is not attained until FS < 1. Therefore, by this method, the FS should not reduce the POF unless the FS is near or below 1.28. A FS threshold of 1.5 would guarantee capturing any change to the POF. A second reason why a threshold FS of 1.5 is sufficient is based upon historical results. Historical results from Phase 1 – Kansas City Levees Feasibility Analysis have shown that for a POF analysis with FS above 1.3, the reliability results were still the maximum (99.8% Reliability). Historical analyses have also shown that POF results did not vary appreciably unless the FS was lower than 1.2. This was largely because the Standard Variation used in analysis was small compared to 0.5, and there were only two variables in the majority of the analyses. Using FS threshold of 1.5 has been shown reliable, theoretically and historically.

A-8.1.5 Reliability Analysis - Existing Conditions Only

For structural features meeting the deterministic strength or stability criteria listed above, a reliability of 99.8% was assigned. For structural features not meeting deterministic strength and stability criterion established above, a risk and reliability analysis was performed. The method adopted for calculating a probability of failure is that outlined for geotechnical engineering in "Factors of Safety and Reliability in Geotechnical

Engineering", by J. Michael Duncan, published in the Journal of Geotechnical and Geoenvironmental Engineering, April 2000. The use of this method provided consistency between the structural and geotechnical analyses.

To produce a probability of failure curve, the critical section of each feature not meeting criteria was analyzed (factor of safety determined) using mean material strengths and/or mean soil properties. Next, each of the parameters was varied to plus and minus one standard deviation from the mean one at a time and the factor of safety was recomputed. The reliability index equation from EM 1110-2-547 was used to determine the reliability of the feature not meeting the factor of safety. The Reliability Index, β, assumes a lognormal distribution, and is relative to a FS equal to 1.0. Assuming the feature started as 100% reliable, the probability of failure was determined by subtracting the reliability from the starting reliability. A 0.2% probability of failure was used as an appropriate non-failure threshold. If a probability of failure greater than 0.2% resulted, then the water elevation was lowered in 1-ft increments and the feature was reanalyzed until the probability of failure obtained was less than 0.2%.

The methods used are appropriate when data is normally distributed, when parameters display a linear relationship, and when degradation over time is not a consideration. Because of the limited availability of data and with no information to suggest otherwise, an assumption of normal distributions for input data is reasonable and consistent with guidance provided in ETL 1110-2-547 (paragraph B-6.c). Examples of non-linear behavior for which the methodology should not be used include overturning stability analysis when the resultant is outside the kern of the base. Examples of degradation over time would include scour around piles, reactive concrete, sliding movement, and deteriorating drainage systems that affect uplift. All available historic data, limited site inspections, and engineering judgment do not show time dependent deterioration of structures to be a concern for the Kansas Citys Levee Systems.

A-8.1.5.1 Risk Calculation

For strength calculations, mean and standard deviation were qualified for the following, concrete and steel strengths. The selected mean and normal standard deviation were based on engineering judgment and information published in Reliability Based Design in Civil Engineering by Milton E. Harr and ETL 1110-2-556.

For stability calculations, mean and standard deviation were qualified for the following, soil unit weight and shear strength. The values were provided by the geotechnical engineers. Varying concrete density had only a minor effect on the factor of safety and therefore was not considered for the risk calculation for this feasibility study.

A-8.1.5.2 Structural Material Properties

For the screening portion of the Kansas Citys Levee Systems feasibility study the following structural properties were used. The American Concrete Institute recommended the use of a 3,000 psi concrete design compressive strength around the 1940s through 1960s, the typical timeframe of construction for most of the levee structures in the study. For earlier concrete strengths little information exists, and 2000 psi concrete was assumed.

Based upon the construction time period (~1940s – 1960s) and the Portland Cement Association's pamphlet, *Engineered Concrete Structures*, 1997, an assumed reinforcing steel minimum design yield strength, Fy, of 40 ksi is used for most computations, unless known or stated otherwise. For earlier structures (1900-1940), the Concrete Reinforcing Steel Institute's *Engineering Data Report 48* suggests 33 ksi steel is typical.

Based on FEMA 310, the mean strength (or expected strength) for Risk and Reliability calculations shall be taken as 125% of the design strength. For reinforced concrete structures Harr suggests a 14% standard deviation.

Concrete Strength Variation (14%)

1940s-1960s:	$-\sigma$ = 3225psi,	μ= 3750psi,	$+\sigma$ = 4275psi (3000 psi min)
1900s-1930s:	$-\sigma$ = 2150psi,	μ = 2500psi,	$+\sigma$ = 2850psi (2000 psi min)

Steel Strength Variation (14%)

1940s-1960s:	$-\sigma$ = 43ksi,	μ = 50ksi,	$+\sigma$ = 57ksi (40 ksi min)
1900s-1930s:	$-\sigma$ = 35.5ksi,	μ = 41.25ksi,	$+\sigma$ = 47.0ksi (33 ksi min)

A-8.1.5.3 Soil Material Properties

The soil properties used for the Kansas Citys Feasibility Study structural calculations can be found in the specific levee unit chapter. In general, the soil material properties used came from historical documentation on the specific levee unit. Soil to structure friction and cohesion interaction was neglected for stability and strength calculations for pile founded floodwalls. However, for gatewells and for spread footing founded floodwalls, this behavior was considered under geotechnical guidance. For material variation, according to ETL 1110-2-561 and ETL 1110-2-556, the following standard deviations are appropriate:

- Soil unit weight: 8%
- Angle of Internal Friction: 10%

For pile capacity, according to ETL 1110-2-561, the following standard deviations were used:

- Compression 25%
- Tension 18%

A-8.1.6 Structural Analysis

The following structural features were analyzed for the Kansas Citys Feasibility Study. Features specific to only one levee unit are mentioned below briefly with feature specifics given in the unit chapters. Features unique to a levee unit and analyzed in a manner different than described below are also more thoroughly discussed in the related levee unit section.

A-8.1.6.1 Floodwalls on Spread Footings

Spread footing floodwalls were analyzed for sliding, bearing capacity and overturning stability, along with wall stem and foundation strengths. Each floodwall cross-section was analyzed using the Corps of Engineers' Computer-Aided Structural Engineering (CASE) project program CTWALL. CTWALL analyzes floodwalls and retaining walls based on *EM 1110-2-2502 Retaining and Flood Walls* (Sep 89). To estimate at-rest pressures using Coulomb's active earth pressure equation, the SMF value in the program was set at 2/3 (0.6667) as indicated by EM 2502 resulting in developed shear strength values assumed to be operative in equilibrium conditions. CTWALL computed a sliding factor of safety, percent base in compression, and maximum bearing pressure. Sliding factors of safety and percent base in compression were then compared to required design minimums. The ratio of bearing pressure to allowable soil bearing capacity (supplied by geotechnical team members) was compared to allowable maximums.

CTWALL output includes a free body diagram detailing the horizontal and vertical forces acting on the wall cross section. These forces were entered into a MathCAD worksheet developed by the Kansas City District to check shear and flexural strengths. The failure of floodwall stems or foundations was based on a capacity/demand ratio of less than one.

To report existing conditions of a floodwall not meeting the minimum strength and stability screening criteria, a reliability analysis was conducted for the floodwall cross section displaying the lowest (controlling) factor of safety and largest COV. Sometimes more than one component can control for development of a POF curve at various water surface elevations. The resulting critical cross section reliability curve was then assigned as the representative curve for the entire reach of floodwall. For example, if a floodwall had 5 different cross sections, Sections A through E, all having the same varied parameters, and section C had the lowest factor of safety, the resulting reliability curve for Section C was used to define the reliability of the entire floodwall.

A-8.1.6.2 Floodwalls on Piles

A Mathcad worksheet was used to perform the static analysis of the floodwalls on piles. The loads applied to the walls were based on EM 1110-2-2502 *Retaining and Flood Walls* (29 Sep 89). This sheet was used to generate the axial, lateral, and bending loads required for input into the Corps' CASE computer program, Pile Group Analysis (CPGA). In addition to the pile cap loading, CPGA required input of pile properties such as type of material (concrete, timber or steel), the shape (square or circular), strength of the material, cross-section and length, fixity of the piles, and soil properties. CPGA in turn determined the combined axial bending forces on the piles. This Program assumes a perfectly rigid pile cap. See closure's structural analysis calculations for levee units with closure to discern how this program's assumption was managed, as well as analysis of torsion in the pile cap.

The individual pile loads output from CPGA were then used to check the pile capacity based on soil and material strength. This included the assessment of the load against the concrete strength capacity by calculating the pile's interaction diagram and also comparing the load with the soil based capacity. The governing factor of safety was then reported.

A-8.1.6.3 Stoplog and Sandbag Closure Structures

For stability, stoplog closure structures were analyzed in a manner similar to floodwalls discussed previously in this chapter. Included in the strength analysis were the stoplog strength, the slots for the stoplogs and the posts, when applicable.

Routine levee inspections of sandbag gaps have revealed no foundation slab issues for the Kansas Citys units. Strength and stability calculations were not performed for sandbag closure structures. If strength or uplift concerns are experienced during flood events, it can reasonably be assumed that flood fighting efforts, through the use of additional sandbags, would be successful in addressing any uplift problems.

A-8.1.6.4 Pump Stations

See pump station specific chapters.

A-8.1.6.5 Gatewells, Reinforced Concrete Boxes, and Drainage Structures

Gatewells and other drainage closure structures were all analyzed in a manner similar to the pump station evaluations. MathCAD worksheets evaluated floatation stability and structural component strengths. Due to the length to width aspect ratios of these structures, plate mechanics were not used. Instead wall and floor component capacities were assessed using one-way beam analysis. The CENWK Local Protection Guidance and EM 1110-2-3104, Appendix B were used to determine the uplift forces for these structures with the exception of vertical resistance mobilized by friction along the exterior face of the structure. Side friction was also considered. An effective lateral load that contributes to side friction was calculated and was used to determine an assumed side friction resistance to uplift. Geotechnical engineers provided the hydraulic grade lines and bottom of blanket elevations at the structure location so that the uplift forces could be calculated. For structures not meeting the screening factors of safety for strength and stability, reliability analysis was conducted.

Strength analysis of each gatewell began with a generalized conservative approach to expedite the process. This initial check considered the load at the base to be acting on the typical section just above the opening for the pipe. This is conservative because at the base an opening typically exists in two walls and all four walls are detailed specially for that reason. Therefore, the load in the initial analysis on the section just above the opening is actually too high. Also, the walls just above the base slab will be supported by that slab and will act in a two-way mode rather than a one-way. If this first check gave acceptable results, no further refinement was necessary.

If the initial check did not produce desirable results, the analysis for the typical box section (just above the opening) was reanalyzed for the actual load acting at that elevation. The walls without openings were then checked as simply supported beams near the base. In cases with large openings, this refinement was often required as the large openings require long walls and resulting gatewells with high aspect ratios. The long walls have openings at the base and do not exist until above the opening while the shorter perpendicular walls form at the base. In such a case, the short walls typically have special reinforcement below the top of the opening in the long walls because they behave more like a simply supported member.

Reinforced concrete boxes were analyzed using the Corps' CASE computer program, for design or investigation of orthogonal culverts (OCRTCUL).

For pipes associated with gatewells, such as RCP, DIP, and CIP, available information and research was used to make recommendations. Generally, the pipe material, the invert elevation, and the size were known, but often little else. In that case, recommendations were made using available information and engineering judgment. Specifics are given in the unit chapters. In addition to a proposed action, detailed inspections will be recommended for all pipes during PED.

A-8.2 STRUCTURAL CONSIDERATIONS IN RAISE ALTERNATIVES

Below is a list of the primary raise alternatives with corresponding structural implications. The list is in order of preference at locations with an existing levee and an existing floodwall.

Existing Levee:

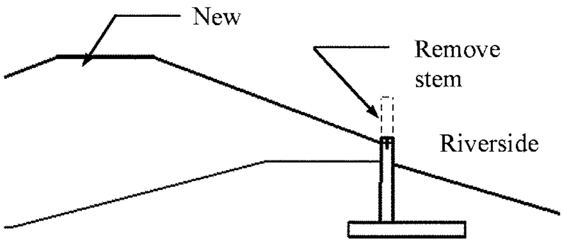
1. Levee Raise
    a. Raises soil and water loads on gatewells and pipes
    b. May require a retaining wall at landside toe
2. T-wall on existing levee
    a. Used to lessen footprint as required by site constraints
    b. Impacts gatewells and pipes by water load only (no additional soil)
3. Floodwall
    a. Required when T-wall on levee requires a stability berm but the real estate is unavailable (reduces global stability concerns by removing some or all of existing levee material)
    b. Will have negligible impact on pipes as existing soil is removed at the maximum fill location

Existing Floodwall:

1. Remove and replace floodwall with levee
    a. Unlikely due to site constraints
    b. Will have significant soil load increase on pipes
2. Use existing floodwall as retaining wall to lessen footprint of landside levee, see Exhibit A-8.1 on the next page
    a. Reduces impact of a landside levee raise
    b. Floodwalls are not typically designed to be loaded from the landside and have little ability to retain landside soil
    c. Requires removal of stem above landside soil to allow for water movement
    d. May have large impact on pipes (large increase in soil loads)

3. Raise existing floodwall
   a. Previous experience has shown that this is more economical than replacement when work is not required on the foundation
   b. Impacts pipes and gatewells due to raised water load
4. Replace existing floodwall with a new floodwall
   a. Nearly equivalent cost to major modification (those requiring foundation work in addition to stem modification) based on Phase 1 estimates

Exhibit A-8.1
Floodwall Acting as Retaining Wall



A-8.3 EXAMPLE CALCULATIONS

Sample calculations can be found in the Supplemental Exhibits section, Exhibits A-8.2 through A-8.5, accompanying this chapter.

A-8.4 SUPPLEMENTAL EXHIBITS

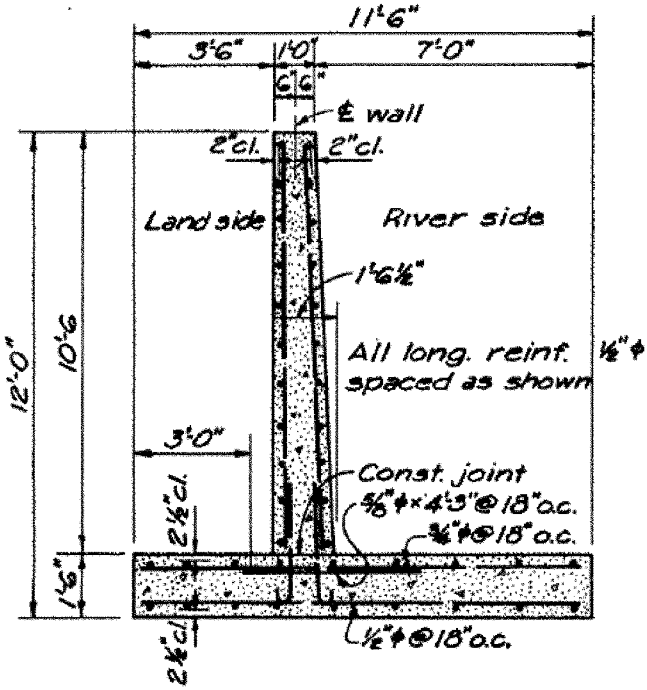
EXHIBIT A-8.2

Spread Footing Floodwall Sample Calculations

US Army Corps of Engineers

CID-MO Flood Unit
Floodwall Analysis for 12' floodwall on spread footings - Page 38 of Record Drawings
Kansas Citys Levees

Comp by:KSM AUG-2011
Chkd by:



Typical Section from 1946 Record Drawings

Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $ksf \equiv \frac{1000lb}{ft^2}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$

CTWALL INPUT FILE NAME: CIDMO12.OUT

Elevation of top of stem (ELTS)...	ELTS := 12ft
Height of stem (HTS)..	HTS := 10.5ft
Thickness of stem (TTS)...	TTS := 1ft
Thickness bottom of stem (TBS)...	TBS := 1.541667ft
Dist. of batter at bot.of stem (TBSR)...	TBSR := 0.541667ft
Depth of heel (THEEL)...	THEEL := 1.5ft
Distance of batter for heel (BTRH)..	BTRH := 0ft
Depth of toe (TTOE)..	TTOE := 1.5ft
Width of toe (TWIDTH)..	TWIDTH := 3.5ft
Distance of batter for toe (BTRT)..	BTRT := 0ft
Width of base (BWIDTH)...	BWIDTH := 11.5ft
Depth of key (HK)...	HK := 0ft
Width of bottom of key (TK)...	TK := 0ft
Dist. of batter at bot. of key (BTRK)..	BTRK := 0ft
Driving side soil elevation (ELSTDS)...	ELSTDS := 5.25ft
Resisting side soil elevation (ELSTRS)...	ELSTRS := 6.25ft
Driving side water elevation (WATELD)..	WATELD := 12ft
Resisting side water elevation (WATELR)....................................	WATELR := 6.25ft

$A := WATELD - ELSTDS$ $G := TBS$

$B := HTS - (ELTS - ELSTDS)$ $H := TWIDTH$

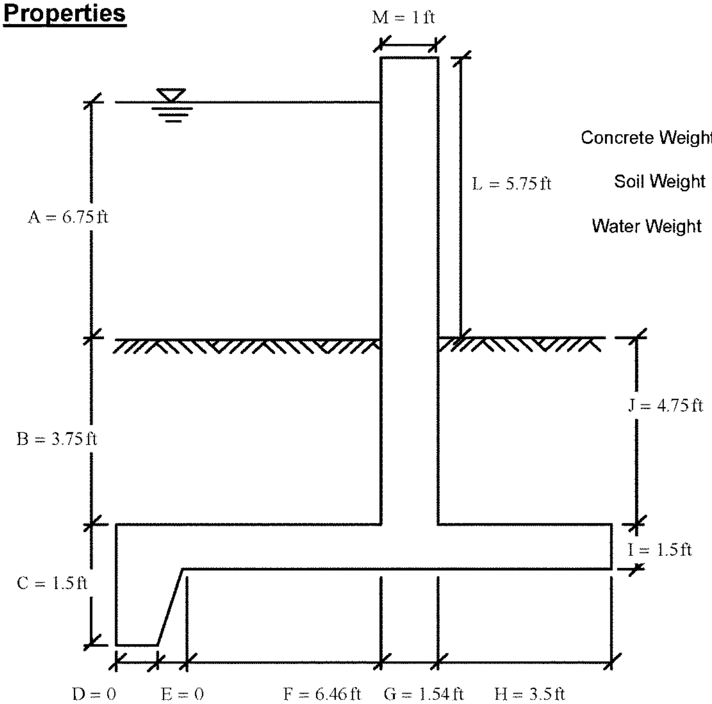
$C := THEEL$ $I := TTOE + BTRT$

$D := TK$ $J := HTS - (ELTS - ELSTRS)$

$E := BTRK$ $L := ELTS - ELSTRS$

$F := BWIDTH - (TWIDTH + TBS + BTRK + TK)$ $M := TTS$

Properties



Concrete Weight $\gamma_c := 150\text{pcf}$

Soil Weight $\gamma := 115\text{pcf}$

Water Weight $\gamma_w := 62.4\text{pcf}$

****************** Echoprint of Input Data ******************

Date: **/08/02 Time: 14.33.26

JANUARY 11, 2008
CIDMO12.DAT

Company name:
USACE
Project name:
KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2
Project location:
CID-MISSOURI
Wall location:
12-FOOT WALL
Computed by: KSM

Structural geometry data:

Elevation of top of stem (ELTS)	=	12.00 ft
Height of stem (HTS)	=	10.50 ft
Thickness top of stem (TTS)	=	1.00 ft
Thickness bottom of stem (TBS)	=	1.54 ft
Dist. of batter at bot. of stem (TBSR)	=	.54 ft
Depth of heel (THEEL)	=	1.50 ft
Distance of batter for heel (BTRH)	=	.00 ft
Depth of toe (TTOE)	=	1.50 ft
Width of toe (TWIDTH)	=	3.50 ft
Distance of batter for toe (BTRT)	=	.00 ft
Width of base (BWIDTH)	=	11.50 ft
Depth of key (HK)	=	.00 ft
Width of bottom of key (TK)	=	.00 ft
Dist. of batter at bot. of key (BTRK)	=	.00 ft

Structure coordinates:

x (ft)	y (ft)
.00	.00
.00	1.50
6.46	1.50
6.46	12.00
7.46	12.00
8.00	1.50
11.50	1.50
11.50	.00

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = .150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
22.00	.000	.110	.115	.00	5.25

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	.00	500.00
2	.00	.00
3	.00	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1493.54	5.25
2	6.46	5.25

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)

```
  22.00     .000      .110      .115     6.25     .00

Resisting side soil profile:

  Soil        x          y
  point      (ft)       (ft)
  ============================
   1          7.76      6.25
   2        507.76      6.25

Foundation property data:
  phi for soil-structure interface =    22.00 (deg)
  c for soil-structure interface   =     .000 (ksf)
  phi for soil-soil interface      =    22.00 (deg)
  c for soil-soil interface        =     .000 (ksf)

Water data:
  Driving side elevation   =    12.00 ft
  Resisting side elevation =     6.25 ft
  Unit weight of water     =    .0624 kcf
  Seepage pressures computed by Line of Creep method.

Minimum required factors of safety:
  Sliding FS   =    1.30
  Overturning  =   25.00% base in compression

Crack options:
  o  Crack *is* down to bottom of heel
  o  Computed cracks *will* be filled with water

User input failure angle data:
  Failure angle of wedge  2 =     .00 deg

Strength mobilization factor =    .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

*Do* iterate in overturning analysis.



***** Summary of Results *****

JANUARY 11, 2008

Project name: KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2

***************     *** Satisfied ***
* Overturning *     Required base in comp. =  25.00 %
***************     Actual base in comp.   =  71.56 %
                    Overturning ratio      =   1.19

Xr (measured from toe) =   2.74 ft
Resultant ratio        =  .2385
Stem ratio             =  .3043
Base pressure at x=   8.23 ft from toe =     .0000 ksf
Base pressure at toe                   =    1.0739 ksf

*** Warning ***  The maximum available shear along the
base of the structure has been exceeded!

***********    *** Not Satisfied ***
* Sliding *    Min. Required =   1.30
***********    Actual FS     =   1.12

  To increase stability try one or a combination
  of the following:
    1.  Increase the base width
    2.  Slope the base of the structure
    3.  Lower the wall base
    4.  Add a key


********************** Output Results **********************


Date: **/08/02                              Time: 14.33.26

JANUARY 11, 2008
CIDMO12.DAT

Company name:
  USACE
```

```
Project name:
  KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2
Project location:
  CID-MISSOURI
Wall location:
  12-FOOT WALL
Computed by: KSM

***************************
**  Overturning Results  **
***************************

Solution converged in  6 iterations.

SMF used to calculate K's =    .6667
Alpha for the SMF         =    .0000
Calculated earth pressure coefficients:
  Driving side at rest K    =    .0000
  Driving side at rest Kc   =    .0000
  Resisting side at rest K  =    .6254
  Resisting side at rest Kc =    .7908
  At-rest K's for resisting side calculated.

Depth of cracking =     5.25 ft
Crack extends to bottom of base of structure.

** Driving side pressures **

  Water pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
        12.00      .0000
          .00      .7488

** Resisting side pressures **

  Water pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
         6.25      .0000
          .00      .5163

  Earth pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
         6.25      .0000
          .00      .1266

** Uplift pressures **

  Water pressures:
     x-coord.   Pressure
       (ft)       (ksf)
    ======================
          .00      .7488
         3.27      .7488
        11.50      .5163

** Forces and moments **
```

Part	Force (kips) Vert.	Force (kips) Horiz.	Mom. Arm (ft)	Moment (ft-k)
Structure:				
Structure weight...........	4.588		-5.16	-23.67
Structure, driving side:				
Moist soil.................	.000		.00	.00
Saturated soil.............	2.786		-8.27	-23.04
Water above structure......	.000		.00	.00
Water above soil...........	2.721		-8.27	-22.50
External vertical loads....	.000		.00	.00
Ext. horz. pressure loads..		.000	.00	.00
Ext. horz. line loads......		.000	.00	.00
Structure, resisting side:				
Moist soil.................	.000		.00	.00
Saturated soil.............	1.979		-1.81	-3.58
Water above structure......	.000		.00	.00
Water above soil...........	.000		.00	.00
Driving side:				
Effective earth loads......		.000	.00	.00
Shear (due to delta).......	.000		.00	.00
Horiz. surcharge effects...		.000	.00	.00
Water loads................		4.493	4.00	17.97
Resisting side:				

Effective earth loads......		-.396	2.08	-.82
Water loads...............		-1.614	2.08	-3.36
Foundation:				
Vertical force on base.....	-4.419		-2.74	12.12
Shear on base.............		-2.484	.00	.00
Uplift....................	-7.654		-6.13	46.89
** Statics Check ** SUMS =	.000	.000		.00

```
Angle of base          =      .00 degrees
Normal force on base =    4.419 kips
Shear force on base  =    2.484 kips
Max. available shear force =    1.785 kips
```

*** Warning *** The maximum available shear along the base of the structure has been exceeded!

```
Base pressure at x=   8.23 ft from toe =    .0000 ksf
Base pressure at toe                   =   1.0739 ksf

Xr (measured from toe) =     2.74 ft
Resultant ratio        =    .2385
Stem ratio             =    .3043
Base in compression    =    71.56 %
Overturning ratio      =     1.19

Volume of concrete =     1.13 cubic yds/ft of wall
```

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

```
***********************
**  Sliding Results  **
***********************
```

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	.000	.000
2	4.493	2.721
3	.000	.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	.0000	.0000		
2			.0000	.7488
2			3.2703	.7488
2			11.5000	.5163
3	.0000	.5163		

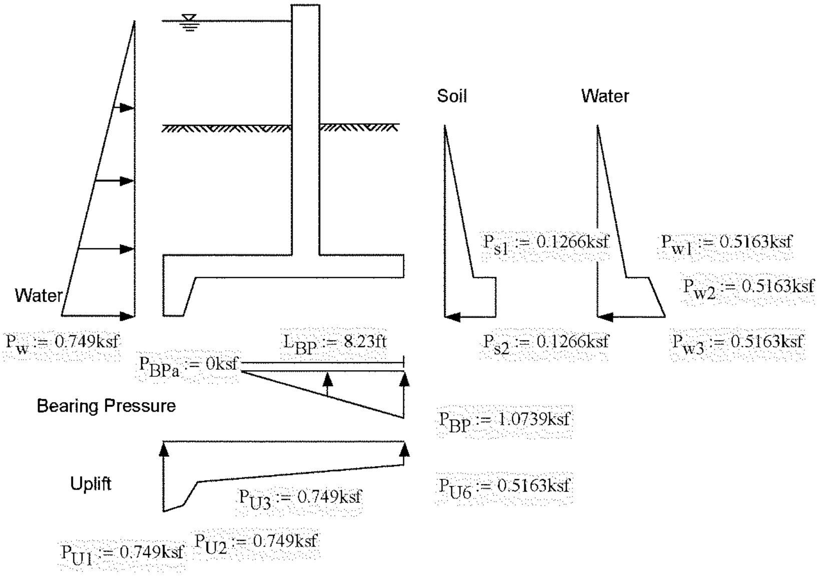
```
Points of sliding plane:
  Point 1 (left),  x =      .00 ft,   y =      .00 ft
  Point 2 (right), x =    11.50 ft,   y =      .00 ft

Depth of cracking =     5.25 ft
Crack extends to bottom of base of structure.
```

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	.000	.000	.000	.000	.000
2	.000	11.500	9.352	11.500	7.655
3	35.090	10.872	3.197	10.872	2.807

Wedge number	Net force (kips)
1	.000
2	-2.897
3	2.897
SUM =	.000

```
+------------------------------+
| Factor of safety =    1.119 |
+------------------------------+
```

Loading from CTWALL Output



Assumptions

- Concrete and reinforcing strengths were not specified in the documents found. However, modifications to the CID-KS unit was under construction/design at the same time as the construction/design of the CID-MO unit. Therefore, it is a reasonable assumption to make that the same material strengths would be specified for CID-MO. The CID-KS design memorandum specifies the concrete strength and reinforcing steel properties as listed here:

  Concrete Properties $f'_c := 3000 \cdot psi$

  Steel Properties $F_y := 36ksi$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

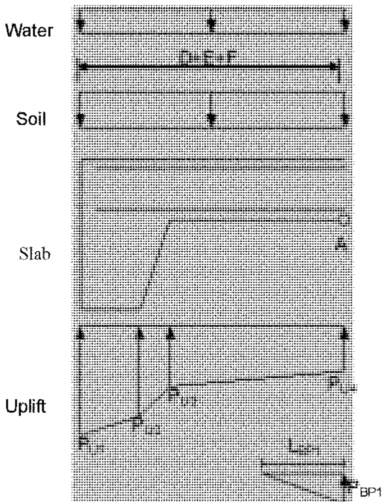
Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Reinforcement Checks

- HEEL

● Location where moment is taken about.



$$W_w := \gamma_w \cdot A \cdot (D + E + F)$$

$$W_w = 2720 \frac{lb}{ft}$$

$$W_s := \gamma \cdot B \cdot (D + E + F)$$

$$W_s = 2785.16 \frac{lb}{ft}$$

$$A_{Heel1} := 0.49 \frac{in^2}{ft} \quad C_{CH1} := 3.375in \qquad d_{H1} := I - C_{CH1}$$

$$A_{Heel2} := 0.133 \frac{in^2}{ft} \quad C_{CH2} := 3.25in \qquad d_{H2} := I - C_{CH2}$$

$$P_{U4} := (P_{U3} - P_{U6}) \cdot \left(\frac{G + H}{F + G + H}\right) + P_{U6}$$

$$P_{U4} = 618.32 \frac{lb}{ft^2}$$

$$L_{BP1} := \text{if}\left[\left[L_{BP} - (G + H)\right] \geq 0ft, \left[L_{BP} - (G + H)\right], 0\right]$$

Bearing Pressure, P_{BP1} acting on the heel at location "A"

$$L_{BP1} = 3.19 ft$$

$$P_{BP1} := \text{if}\left[L_{BP} < (D + E + F + G + H), \frac{L_{BP1}}{L_{BP}} \cdot P_{BP}, P_{BPa} + \frac{L_{BP1} \cdot (P_{BP} - P_{BPa})}{L_{BP}}\right.$$

$$P_{BP1} = 416 \frac{lb}{ft^2}$$

Slab Centroid

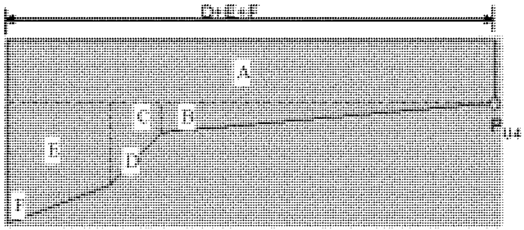
$$X_H := \frac{(D + E + F) \cdot I \cdot \left(\frac{D + E + F}{2}\right) + (C - I) \cdot D \cdot \left(E + F + \frac{D}{2}\right) + \left[\frac{(C - I) \cdot E}{2}\right] \cdot \left(F + \frac{2 \cdot E}{3}\right)}{(D + E + F) \cdot I + (C - I) \cdot D + \frac{(C - I) \cdot E}{2}} \qquad X_H = 3.23 ft$$

- HEEL check (cont'd)

Slab Weight

$$W_H := \left[(D + E + F)\cdot I + (C - I)\cdot D + \frac{(C - I)\cdot E}{2}\right]\cdot 150pcf \qquad W_H = 1453\,\frac{lb}{ft}$$

Uplift Centroid



AREA

$$A_A := (D + E + F)\cdot P_{U4}$$

$$A_B := \frac{F\cdot\left(P_{U3} - P_{U4}\right)}{2}$$

$$A_C := E\cdot\left(P_{U3} - P_{U4}\right)$$

$$A_D := \frac{E\cdot\left(P_{U2} - P_{U3}\right)}{2}$$

$$A_E := D\cdot\left(P_{U2} - P_{U4}\right)$$

$$A_F := \frac{D\cdot\left(P_{U1} - P_{U2}\right)}{2}$$

CENTROID

$$C_A := \frac{D + E + F}{2}$$

$$C_B := \frac{2\cdot F}{3}$$

$$C_C := F + \frac{E}{2}$$

$$C_D := F + \frac{2\cdot E}{3}$$

$$C_E := \left(\frac{D}{2} + E + F\right)$$

$$C_F := \left(\frac{2\cdot D}{3} + E + F\right)$$

$$X_U := \frac{A_A\cdot C_A + A_B\cdot C_B + A_C\cdot C_C + A_D\cdot C_D + A_E\cdot C_E + A_F\cdot C_F}{A_A + A_B + A_C + A_D + A_E + A_F} \qquad X_U = 3.33\,ft$$

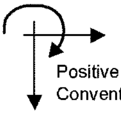
Uplift on Heel $\quad W_U := A_A + A_B + A_C + A_D + A_E + A_F \qquad W_U = 4415\,\frac{lb}{ft}$

- HEEL check (cont'd)

Loading:

Bending

$$M_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[W_U \cdot X_U + \frac{P_{BP1} \cdot L_{BP1}}{2} \cdot \frac{L_{BP1}}{3} - \left[(W_w + W_s) \cdot \left(\frac{D + E + F}{2} \right) + W_H \cdot X_H \right] \right]$$



Positive Sign Convention

$$M_H = -7053.44 \cdot \frac{ft \cdot lb}{ft} \qquad M_{uH} := |M_H|$$

Note:
IF: M_H is < 0
THEN: Steel in top of heel is in tension

Shear

$$V_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \left(W_w + W_s + W_H - W_U - \frac{P_{BP1} \cdot L_{BP1}}{2} \right) \qquad M_{uH} = 7.1 \cdot \frac{kip \cdot ft}{ft}$$

$$V_H = 1880.01 \frac{lb}{ft} \qquad V_{uH} := |V_H| \qquad V_{uH} = 1.9 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

$$A_{s1} := A_{Heel1} \qquad A_{s2} := A_{Heel2}$$

$$b := 12 \frac{in}{ft} \qquad b := 12 \frac{in}{ft}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 0.58 \cdot in \qquad a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \qquad a2 = 0.16 \cdot in$$

$$\phi M_{H1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{H1} - \frac{a1}{2} \right) \qquad \phi M_{H2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{H2} - \frac{a2}{2} \right)$$

$$\phi M_{H1} = 21.08 \cdot \frac{kip \cdot ft}{ft} \qquad \phi M_{H2} = 5.85 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_H := if(M_H > 0, \phi M_{H2}, \phi M_{H1}) \qquad \phi M_H = 21.08 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{H2} \cdot \sqrt{f'_c \cdot psi}$$

$$\phi V_{c1} = 19225 \frac{lb}{ft} \qquad \phi V_{c2} = 19389 \frac{lb}{ft}$$

$$\phi V_H := if(M_H > 0, \phi V_{c2}, \phi V_{c1}) \qquad \phi V_H = 19.23 \cdot \frac{kip}{ft}$$

- HEEL check (cont'd)

Factors of Safety

Bending $\phi M_H = 21.08 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_1 := \frac{\phi M_H}{M_{uH}}$ $\quad FS_1 = 2.99$

$M_{uH} = 7.05 \cdot \frac{kip \cdot ft}{ft}$

$Check1 := if(\phi M_H > 1.5 M_{uH}, "OKAY", "NO\ GOOD")$ $\quad Check1 = "OKAY"$

Shear $\phi V_H = 19.23 \cdot \frac{kip}{ft}$ $\quad FS_2 := \frac{\phi V_H}{V_{uH}}$

$V_{uH} = 1.88 \cdot \frac{kip}{ft}$ $\quad FS_2 = 10.23$

$Check2 := if(\phi V_H > 1.5 V_{uH}, "OKAY", "NO\ GOOD")$ $\quad Check2 = "OKAY"$

Controlling Factor of Safety

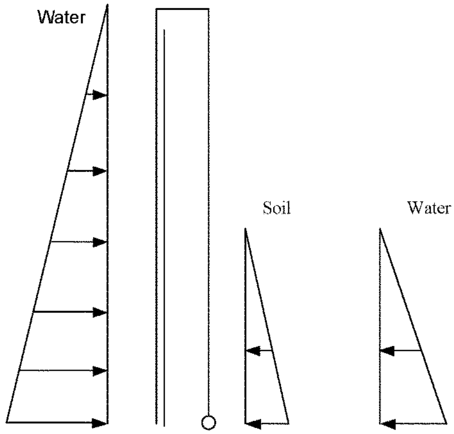
$FS_H := min(FS_1, FS_2)$ $\quad FS_H = 2.99$

Controlling Mechanism

$Comment_H := if(FS_1 > FS_2, "Shear\ in\ Heel", if(M_H > 0, "Flexural\ Bottom\ Steel\ in\ Heel", "Flexural\ Top\ Steel\ in\ Heel"))$

$Comment_H = "Flexural\ Top\ Steel\ in\ Heel"$

- STEM Check



Note:
P_{ss} = Soil pressure present at base of stem
P_{ws2} = Water pressure present at base of stem

$P_{ws1} := (A + B)\cdot\gamma_w$

$P_{ws1} = 655 \frac{lb}{ft^2}$

$A_{Stem1} := 0.35 \frac{in^2}{ft}$ $A_{Stem2} := 0.2 \frac{in^2}{ft}$

$C_{CS1} := 2.8125in$ $C_{CS2} := 2.75in$

$d_{S1} := G - C_{CS1}$ $d_{S2} := G - C_{CS2}$

$P_{ss} := \frac{J}{I+J}\cdot P_{s1}$ $P_{ws2} := \frac{J}{I+J}\cdot P_{w1}$

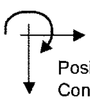
$P_{ss} = 96.22 \frac{lb}{ft^2}$ $P_{ws2} = 392.39 \frac{lb}{ft^2}$

Loading:

Bending

$$M_S := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A+B)}{2}\cdot\frac{A+B}{3} - \frac{(P_{ss}+P_{ws2})\cdot(J)}{2}\cdot\frac{J}{3}\right]$$

Note:
IF: M_S is > 0
THEN: Riverside steel is in tension



$M_S = 10201.9\cdot\frac{ft\cdot lb}{ft}$ $M_{uS} := |M_S|$ $M_{uS} = 10.2\cdot\frac{kip\cdot ft}{ft}$

Shear

$$V_S := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A+B)}{2} - \frac{(P_{ss}+P_{ws2})\cdot(J)}{2}\right]$$

$V_S = 2.28\cdot\frac{kip}{ft}$ $V_{uS} := |V_S|$ $V_{uS} = 2.3\cdot\frac{kip}{ft}$

- STEM Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$ $\qquad$ $A_{s2} := A_{Stem2}$

$b := 12\frac{in}{ft}$ $\qquad$ $b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85f'_c \cdot b}$ $\quad a1 = 0.41 \cdot in$ $\qquad$ $a2 := \frac{A_{s2} \cdot F_y}{0.85f'_c \cdot b}$ $\quad a2 = 0.24 \cdot in$

$\phi M_{S1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{S1} - \frac{a1}{2}\right)$ $\qquad$ $\phi M_{S2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{S2} - \frac{a2}{2}\right)$

$\phi M_{S1} = 16.26 \cdot \frac{kip \cdot ft}{ft}$ $\qquad$ $\phi M_{S2} = 9.38 \cdot \frac{kip \cdot ft}{ft}$

$\phi M_S := if\left(M_S > 0, \phi M_{S1}, \phi M_{S2}\right)$ $\qquad$ $\phi M_S = 16.26 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{S1} \cdot \sqrt{f'_c \cdot psi}$ $\qquad$ $\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{S2} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c1} = 20622 \frac{lb}{ft}$ $\qquad$ $\phi V_{c2} = 20704 \frac{lb}{ft}$

$\phi V_S := if\left(M_S > 0, \phi V_{c1}, \phi V_{c2}\right)$ $\qquad$ $\phi V_S = 20.62 \cdot \frac{kip}{ft}$

Factors of Safety

Bending

$\phi M_S = 16.26 \cdot \frac{kip \cdot ft}{ft}$

$M_{uS} = 10.2 \cdot \frac{kip \cdot ft}{ft}$

$FS_3 := \frac{\phi M_S}{M_{uS}}$ $\qquad$ $FS_3 = 1.59$

Check3 := if($\phi M_S > 1.5M_{uS}$, "OKAY", "NO GOOD") $\qquad$ Check3 = "OKAY"

Shear

$\phi V_S = 20.62 \cdot \frac{kip}{ft}$

$V_{nS} = 2.28 \cdot \frac{kip}{ft}$

$FS_4 := \frac{\phi V_S}{V_{uS}}$ $\qquad$ $FS_4 = 9.05$

Check4 := if($\phi V_S > 1.5V_{uS}$, "OKAY", "NO GOOD") $\qquad$ Check4 = "OKAY"

Controlling Factor of Safety

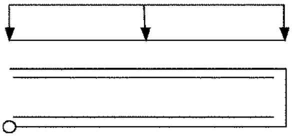
$FS_S := \min\left(FS_3, FS_4\right)$ $\qquad$ $FS_S = 1.59$

Controlling Mechanism

- TOE Check

Soil

Slab



$W_s := \gamma \cdot H \cdot (J)$ $\quad W_s = 1911.87 \frac{lb}{ft}$

$A_{Toe1} := 0.29 \frac{in^2}{ft}$ $\quad C_{CT1} := 3.375in$ $\quad d_{T1} := I - C_{CT1}$

$A_{Toe2} := 0.133 \frac{in^2}{ft}$ $\quad C_{CT2} := 3.25in$ $\quad d_{T1} = 14.63 \cdot in$

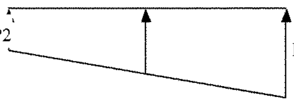
$L_{BP2} := if(H \le L_{BP}, H, L_{BP})$

$L_{BP2} = 3.5ft$

$d_{T2} := I - C_{CT2}$

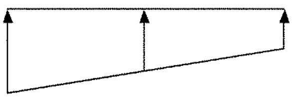
$d_{T2} = 14.75 \cdot in$

Bearing Pressure P_{BP2}



$P_{BP} = 1073.9 \frac{lb}{ft^2}$

$$P_{BP2} := if\left[L_{BP} < (D + E + F + G + H), P_{BP} - P_{BP} \cdot \frac{L_{BP2}}{L_{BP}}, P_{BP} - L_{BP2} \cdot \left(\frac{P_{BP} - P_{BPa}}{L_{BP}}\right)\right] = 617.2 \frac{lb}{ft^2}$$



$P_{U6} = 516.3 \frac{lb}{ft^2}$

Uplift

$$P_{U5} := \left[\left[(P_{U3} - P_{U6}) \cdot \frac{H}{F + G + H} + P_{U6}\right]\right]$$

$$P_{U5} = 587.12 \frac{lb}{ft^2}$$

Bearing Pressure

$$L_{bp} := \frac{(L_{BP2} \cdot P_{BP2}) \cdot \left(\frac{L_{BP2}}{2}\right) + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2} \cdot \left(\frac{2L_{BP}}{3}\right)}{L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2}} \qquad L_{bp} = 2.76ft$$

$$W_{bp} := L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2} \qquad W_{bp} = 2959 \frac{lb}{ft}$$

Uplift

$$L_u := \frac{H \cdot P_{U6} \cdot \frac{H}{2} + \frac{(P_{U5} - P_{U6}) \cdot H}{2} \cdot \frac{H}{3}}{H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2}} \qquad L_u = 1.71ft$$

$$W_u := H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2} \qquad W_u = 1930.99 \frac{lb}{ft}$$

Toe Check (continued)

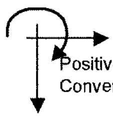
Loading:

Bending

$$M_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[\left[W_s + (H \cdot I) \cdot \gamma_c \right] \cdot \frac{H}{2} - \left(W_{bp} \cdot L_{bp} + W_u \cdot L_u \right) \right]$$

Note:
IF: $M_T < 0$
THEN: Bottom steel is in tension

$$M_T = -6.7 \cdot \frac{kip \cdot ft}{ft} \qquad M_{uT} := |M_T| \qquad M_{uT} = 6.75 \cdot \frac{kip \cdot ft}{ft}$$



Positive Sign Convention

Shear

$$V_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[W_s + (H \cdot I) \cdot \gamma_c - W_{bp} - W_u \right]$$

$$V_T = -2.19 \cdot \frac{kip}{ft} \qquad V_{uT} := |V_T| \qquad V_{uT} = 2.19 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

$$A_{s1} := A_{Toe1} \qquad A_{s2} := A_{Toe2}$$

$$b := 12 \frac{in}{ft} \qquad b := 12 \frac{in}{ft}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 0.34 \cdot in \qquad a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \qquad a2 = 0.16 \cdot in$$

$$\phi M_{T1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{T1} - \frac{a1}{2} \right) \qquad \phi M_{T2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{T2} - \frac{a2}{2} \right)$$

$$\phi M_{T1} = 12.58 \cdot \frac{kip \cdot ft}{ft} \qquad \phi M_{T2} = 5.85 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_T := \text{if}\left(M_T > 0, \phi M_{T1}, \phi M_{T2} \right) \qquad \phi M_T = 5.85 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{T1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{T2} \cdot \sqrt{f'_c \cdot psi}$$

$$\phi V_{c1} = 19225 \frac{lb}{ft} \qquad \phi V_{c2} = 19389 \frac{lb}{ft}$$

$$\phi V_T := \text{if}\left(M_{uT} > 0, \phi V_{c1}, \phi V_{c2} \right) \qquad \phi V_T = 19.23 \cdot \frac{kip}{ft}$$

Toe Check (continued)

Factors of Safety

Bending

$\phi M_T = 5.85 \cdot \frac{kip \cdot ft}{ft}$

$M_{uT} = 6.75 \cdot \frac{kip \cdot ft}{ft}$

$FS_5 := \frac{\phi M_T}{M_{uT}}$

$FS_5 = 0.87$

$Check5 := if(\phi M_T > 1.5 \cdot M_{uT}, "OKAY", "NO GOOD")$

$Check5 = "NO GOOD"$

Shear

$\phi V_T = 19.23 \cdot \frac{kip}{ft}$

$V_{uT} = 2.19 \cdot \frac{kip}{ft}$

$FS_6 := \frac{\phi V_T}{V_{uT}}$

$FS_6 = 8.77$

$Check6 := if(\phi V_T > 1.5 V_{uT}, "OKAY", "NO GOOD")$

$Check6 = "OKAY"$

Controlling Factor of Safety

$FS_T := min(FS_5, FS_6)$

$FS_T = 0.87$

Controlling Mechanism

$Comment_T := if(FS_5 > FS_6, "Shear in Heel", if(M_T > 0, "Flexural Top Steel in Toe", "Flexural Bottom Steel in Toe"))$

$Comment_T = "Flexural Bottom Steel in Toe"$

Overall Factor of Safety (without considering multiple layers of steel)

Factor of Safety

$$FoS := \min(FS_H, FS_S, FS_T)$$

Limiting Mechanism

$$Mechanism := Comment_H$$

$$Mechanism := if(FoS = FS_S, Comment_S, Mechanism)$$

$$Mechanism := if(FoS = FS_T, Comment_T, Mechanism)$$

$$FoS = 0.87$$

Mechanism = "Flexural Bottom Steel in Toe"

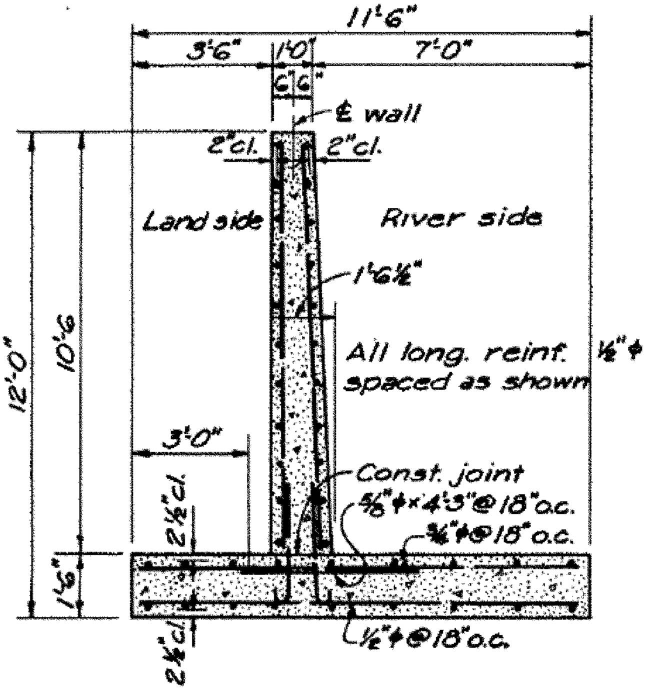
It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis. If the factor of safety is lower than 1.5 a probablility analysis is required.

US Army Corps of Engineers

CID-MO Flood Unit
Floodwall Analysis for 12' floodwall on spread footings - Page 38 of Record Drawings
Kansas Citys Levees

Comp by:KSM 3-18-08
Chkd by:

water 1ft down from top for POF analysis



Typical Section from 1946 Record Drawings

Variables

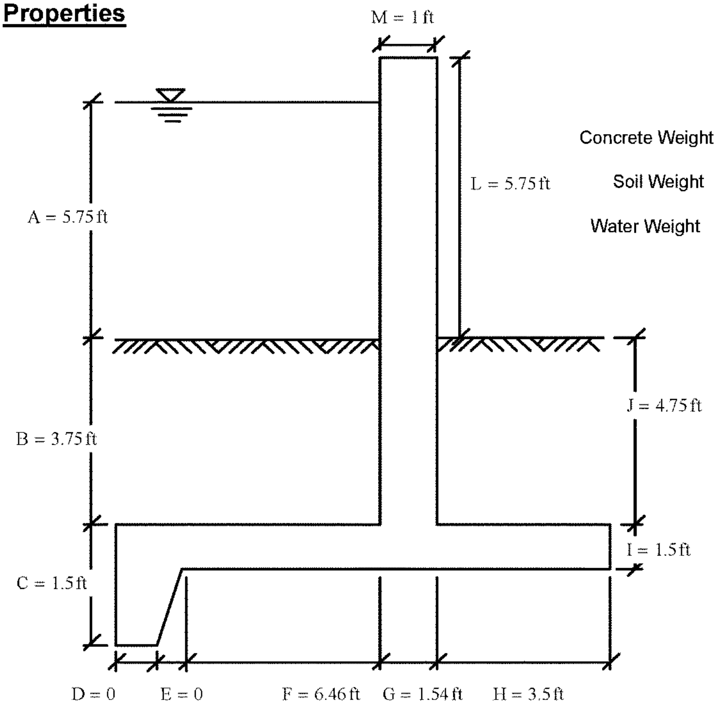
$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $ksf \equiv \frac{1000lb}{ft^2}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$

CTWALL INPUT FILE NAME: CIDMO121.OUT

Elevation of top of stem (ELTS)..	ELTS := 12ft
Height of stem (HTS)..	HTS := 10.5ft
Thickness of stem (TTS)...	TTS := 1ft
Thickness bottom of stem (TBS)..	TBS := 1.541667ft
Dist. of batter at bot.of stem (TBSR)..	TBSR := 0.541667ft
Depth of heel (THEEL)...	THEEL := 1.5ft
Distance of batter for heel (BTRH)...	BTRH := 0ft
Depth of toe (TTOE)..	TTOE := 1.5ft
Width of toe (TWIDTH)..	TWIDTH := 3.5ft
Distance of batter for toe (BTRT)..	BTRT := 0ft
Width of base (BWIDTH)..	BWIDTH := 11.5ft
Depth of key (HK)..	HK := 0ft
Width of bottom of key (TK)..	TK := 0ft
Dist. of batter at bot. of key (BTRK)...	BTRK := 0ft
Driving side soil elevation (ELSTDS)...	ELSTDS := 5.25ft
Resisting side soil elevation (ELSTRS)...	ELSTRS := 6.25ft
Driving side water elevation (WATELD)..	WATELD := 11ft
Resisting side water elevation (WATELR)....................................	WATELR := 6.25ft

A := WATELD − ELSTDS	G := TBS
B := HTS − (ELTS − ELSTDS)	H := TWIDTH
C := THEEL	I := TTOE + BTRT
D := TK	J := HTS − (ELTS − ELSTRS)
E := BTRK	L := ELTS − ELSTRS
F := BWIDTH − (TWIDTH + TBS + BTRK + TK)	M := TTS

Properties



Concrete Weight $\gamma_c := 150pcf$

Soil Weight $\gamma := 115pcf$

Water Weight $\gamma_w := 62.4pcf$

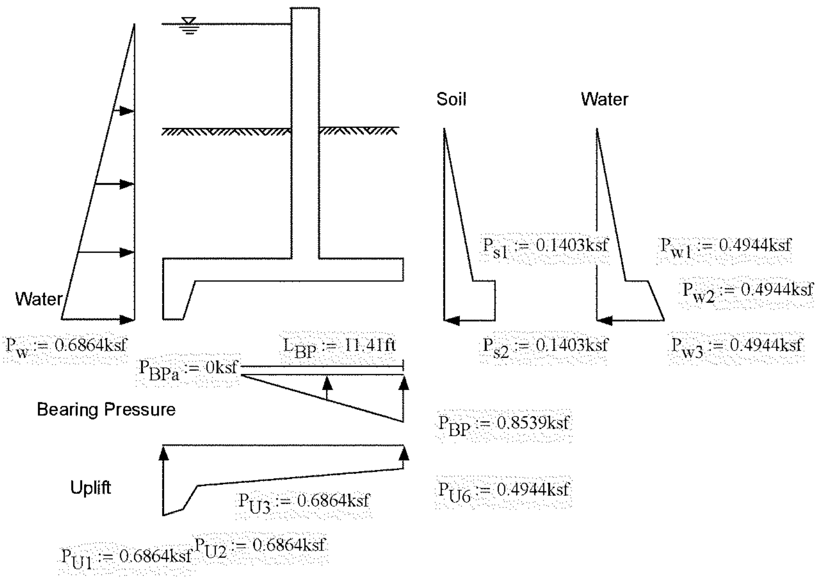
```
****************** Echoprint of Input Data ******************

Date: **/08/02                              Time: 15.43.39

JANUARY 11, 2008
CIDMO12.DAT

Company name:
  USACE
Project name:
  KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2
Project location:
  CID-MISSOURI
Wall location:
  12-FOOT WALL
Computed by: KSM

Structural geometry data:
  Elevation of top of stem (ELTS)        =    12.00 ft
  Height of stem (HTS)                   =    10.50 ft
  Thickness top of stem (TTS)            =     1.00 ft
  Thickness bottom of stem (TBS)         =     1.54 ft
  Dist. of batter at bot. of stem (TBSR)=       .54 ft
  Depth of heel (THEEL)                  =     1.50 ft
  Distance of batter for heel (BTRH)     =      .00 ft
  Depth of toe (TTOE)                    =     1.50 ft
  Width of toe (TWIDTH)                  =     3.50 ft
  Distance of batter for toe (BTRT)      =      .00 ft
  Width of base (BWIDTH)                 =    11.50 ft
  Depth of key (HK)                      =      .00 ft
  Width of bottom of key (TK)            =      .00 ft
  Dist. of batter at bot. of key (BTRK) =      .00 ft

Structure coordinates:

   x (ft)    y (ft)
  ================
      .00       .00
      .00      1.50
     6.46      1.50
     6.46     12.00
     7.46     12.00
     8.00      1.50
    11.50      1.50
    11.50       .00

NOTE: X=0 is located at the left-hand side
      of the structure.  The Y values correspond
      to the actual elevation used.

Structural property data:
  Unit weight of concrete =     .150 kcf

Driving side soil property data:

                          Moist   Saturated            Elev.
    Phi         c       Unit wt.  unit wt.   Delta    soil
   (deg)      (ksf)      (kcf)     (kcf)     (deg)    (ft)
=======================================================
   22.00      .000       .110      .115       .00     5.25

Driving side soil geometry:

  Soil       Batter    Distance
  point     (in:1ft)     (ft)
  ===========================
  1            .00    500.00
  2            .00       .00
  3            .00    500.00

Driving side soil profile:

  Soil         x           y
  point       (ft)        (ft)
  ===========================
  1       -1493.54       5.25
  2           6.46       5.25

Resisting side soil property data:

                          Moist   Saturated  Elev.
    Phi         c       Unit wt.  unit wt.   soil     Batter
   (deg)      (ksf)      (kcf)     (kcf)     (ft)    (in:1ft)
=========================================================
```

```
   22.00      .000       .110       .115      6.25      .00

Resisting side soil profile:

  Soil        x          y
  point      (ft)       (ft)
  ============================
   1          7.76      6.25
   2        507.76      6.25

Foundation property data:
  phi for soil-structure interface =    22.00 (deg)
  c for soil-structure interface   =     .000 (ksf)
  phi for soil-soil interface      =    22.00 (deg)
  c for soil-soil interface        =     .000 (ksf)

Water data:
  Driving side elevation   =    11.00 ft
  Resisting side elevation =     6.25 ft
  Unit weight of water     =    .0624 kcf
  Seepage pressures computed by Line of Creep method.

Minimum required factors of safety:
  Sliding FS   =     1.30
  Overturning  =   25.00% base in compression

Crack options:
  o  Crack *is* down to bottom of heel
  o  Computed cracks *will* be filled with water

User input failure angle data:
  Failure angle of wedge  2 =      .00 deg

Strength mobilization factor =     .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

*Do* iterate in overturning analysis.



***** Summary of Results *****

JANUARY 11, 2008

Project name: KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2

***************     *** Satisfied ***
* Overturning *     Required base in comp. =  25.00 %
***************     Actual base in comp.   =  99.23 %
                    Overturning ratio      =   1.34

Xr (measured from toe) =   3.80 ft
Resultant ratio        =  .3308
Stem ratio             =  .3043
Base pressure at x=  11.41 ft from toe =     .0000 ksf
Base pressure at toe                   =     .8539 ksf

***********    *** Satisfied ***
* Sliding *    Min. Required =   1.30
***********    Actual FS     =   1.75


********************** Output Results *********************


Date: **/08/02                               Time: 15.43.39

JANUARY 11, 2008
CIDMO12.DAT

Company name:
  USACE
Project name:
  KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2
Project location:
  CID-MISSOURI
Wall location:
  12-FOOT WALL
Computed by: KSM

***************************
**  Overturning Results  **
```

```
**************************

Solution converged in  4 iterations.

SMF used to calculate K's =    .6667
Alpha for the SMF          =    .0000
Calculated earth pressure coefficients:
  Driving side at rest K    =    .0000
  Driving side at rest Kc   =    .0000
  Resisting side at rest K  =    .6254
  Resisting side at rest Kc =    .7908
  At-rest K's for resisting side calculated.

Depth of cracking =     5.25 ft
Crack extends to bottom of base of structure.

** Driving side pressures **

  Water pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
       11.00       .0000
         .00       .6864

** Resisting side pressures **

  Water pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
        6.25       .0000
         .00       .4944

  Earth pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
        6.25       .0000
         .00       .1403

** Uplift pressures **

  Water pressures:
     X-coord.   Pressure
      (ft)        (ksf)
    ======================
         .00       .6864
         .09       .6864
       11.50       .4944

** Forces and moments **

=========================================================================
              Part              |  Force (kips)      | Mom. Arm  |  Moment |
                                | Vert.   |   Horiz.|   (ft)    |  (ft-k) |
=========================================================================
Structure:
  Structure weight...........   4.588                  -5.16     -23.67
Structure, driving side:
  Moist soil.................    .000                    .00        .00
  Saturated soil.............   2.786                  -8.27     -23.04
  Water above structure......    .000                    .00        .00
  Water above soil...........   2.318                  -8.27     -19.17
  External vertical loads....    .000                    .00        .00
  Ext. horz. pressure loads..              .000          .00        .00
  Ext. horz. line loads......              .000          .00        .00
Structure, resisting side:
  Moist soil.................    .000                    .00        .00
  Saturated soil.............   1.979                  -1.81      -3.58
  Water above structure......    .000                    .00        .00
  Water above soil...........    .000                    .00        .00
Driving side:
  Effective earth loads......              .000          .00        .00
  Shear (due to delta).......    .000                    .00        .00
  Horiz. surcharge effects...              .000          .00        .00
  Water loads................             3.775         3.67      13.84
Resisting side:
  Effective earth loads......             -.439         2.08       -.91
  Water loads................            -1.545         2.08      -3.22
Foundation:
  Vertical force on base.....  -4.872                  -3.80      18.53
  Shear on base..............            -1.792          .00        .00
  Uplift.....................  -6.798                  -6.06      41.22
=========================================================================
** Statics Check **   SUMS =     .000      .000                     .00

Angle of base        =      .00 degrees
```

```
Normal force on base =    4.872 kips
Shear force on base  =    1.792 kips
Max. available shear force =    1.968 kips

Base pressure at x=  11.41 ft from toe =    .0000 ksf
Base pressure at toe                   =    .8539 ksf

Xr (measured from toe) =     3.80 ft
Resultant ratio        =    .3308
Stem ratio             =    .3043
Base in compression    =    99.23 %
Overturning ratio      =     1.34

Volume of concrete =     1.13 cubic yds/ft of wall

NOTE:  The engineer shall verify that the computed
bearing pressures below the wall do not exceed the
allowable foundation bearing pressure, or, perform a
bearing capacity analysis using the program CBEAR.
Also, the engineer shall verify that the base pressures
do not result in excessive differential settlement of
the wall foundation.



***********************
**  Sliding Results  **
***********************

Solution converged.  Summation of forces = 0.

          Horizontal   Vertical
  Wedge      Loads       Loads
  Number    (kips)      (kips)
  ==================================
     1        .000        .000
     2       3.775       2.318
     3        .000        .000

  Water pressures on wedges:

             Top      Bottom
    Wedge   press.   press.   x-coord.     press.
    number  (ksf)    (ksf)     (ft)        (ksf)
  ===================================================
      1     .0000     .0000
      2                          .0000       .6864
      2                          .0890       .6864
      2                        11.5000       .4944
      3     .0000     .4944

Points of sliding plane:
  Point 1 (left),  x =      .00 ft,   y =      .00 ft
  Point 2 (right), x =    11.50 ft,   y =      .00 ft

Depth of cracking =     5.25 ft
Crack extends to bottom of base of structure.

          Failure   Total    Weight    Submerged   Uplift
  Wedge    angle    length  of wedge    length     force
  number   (deg)     (ft)    (kips)      (ft)      (kips)
  ========================================================
     1      .000      .000      .000      .000      .000
     2      .000    11.500     9.352    11.500     6.798
     3    38.558    10.027     2.818    10.027     2.479

  Wedge     Net force
  number     (kips)
  ===================
     1         .000
     2       -2.652
     3        2.652
  ===================
     SUM =     .000

+-----------------------------+
| Factor of safety =    1.753 |
+-----------------------------+
```

Loading from CTWALL Output



Assumptions

- Concrete and reinforcing strengths were not specified in the documents found. However, modifications to the CID-KS unit was under construction/design at the same time as the construction/design of the CID-MO unit. Therefore, it is a reasonable assumption to make that the same material strengths would be specified for CID-MO. The CID-KS design memorandum specifies the concrete strength and reinforcing steel properties as listed here:

  Concrete Properties $f'_c := 3000 \cdot psi$

  Steel Properties $F_y := 36ksi$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength	$\phi_V := 1.0$	Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.
Flexural Strength	$\phi_B := 1.0$	

Load Factors

Dead and Live Load Factor	$\gamma_L := 1.0$	Load Multiplication Factor EM 1110-2-2104 (3-1)
Hydraulic Load Factor	$\gamma_H := 1.0$	Hydraulic Factor EM 1110-2-2104 (3-2)
Extreme Case Factor	$\gamma_X := 1.0$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Reinforcement Checks

- HEEL

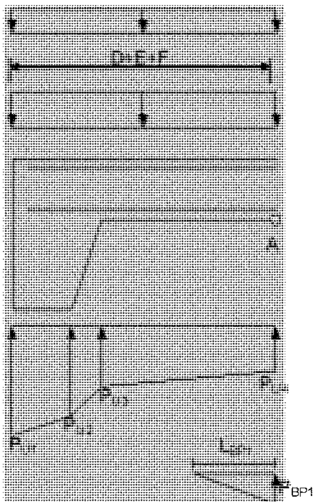
Location where moment is taken about.

Water

Soil

Slab

Uplift



Bearing Pressure, P_{BP1} acting on the heel at location "A"

$W_w := \gamma_w \cdot A \cdot (D + E + F)$

$W_w = 2317 \frac{lb}{ft}$

$W_s := \gamma \cdot B \cdot (D + E + F)$

$W_s = 2785.16 \frac{lb}{ft}$

$A_{Heel1} := 0.49 \frac{in^2}{ft}$ $C_{CH1} := 3.375 in$ $d_{H1} := I - C_{CH1}$

$A_{Heel2} := 0.133 \frac{in^2}{ft}$ $C_{CH2} := 3.25 in$ $d_{H2} := I - C_{CH2}$

$$P_{U4} := (P_{U3} - P_{U6}) \cdot \left(\frac{G + H}{F + G + H}\right) + P_{U6}$$

$P_{U4} = 578.57 \frac{lb}{ft^2}$

$$L_{BP1} := \text{if}\left[\left[L_{BP} - (G + H)\right] \ge 0ft, \left[L_{BP} - (G + H)\right], 0\right]$$

$L_{BP1} = 6.37 ft$

$$P_{BP1} := \text{if}\left[L_{BP} < (D + E + F + G + H), \frac{L_{BP1}}{L_{BP}} \cdot P_{BP}, P_{BPa} + \frac{L_{BP1} \cdot (P_{BP} - P_{BPa})}{L_{BP}}\right.$$

$P_{BP1} = 477 \frac{lb}{ft^2}$

Slab Centroid

$$X_H := \frac{(D + E + F) \cdot I \cdot \left(\frac{D + E + F}{2}\right) + (C - I) \cdot D \cdot \left(E + F + \frac{D}{2}\right) + \left[\frac{(C - I) \cdot E}{2}\right] \cdot \left(F + \frac{2 \cdot E}{3}\right)}{(D + E + F) \cdot I + (C - I) \cdot D + \frac{(C - I) \cdot E}{2}}$$

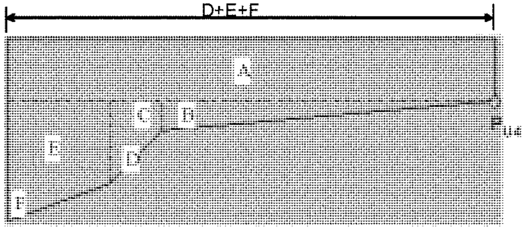
$X_H = 3.23 ft$

- HEEL check (cont'd)

Slab Weight

$$W_H := \left[(D+E+F)\cdot I + (C-I)\cdot D + \frac{(C-I)\cdot E}{2}\right]\cdot 150\text{pcf} \qquad W_H = 1453\,\frac{\text{lb}}{\text{ft}}$$

Uplift Centroid



AREA

$$A_A := (D+E+F)\cdot P_{U4}$$

$$A_B := \frac{F\cdot\left(P_{U3}-P_{U4}\right)}{2}$$

$$A_C := E\cdot\left(P_{U3}-P_{U4}\right)$$

$$A_D := \frac{E\cdot\left(P_{U2}-P_{U3}\right)}{2}$$

$$A_E := D\cdot\left(P_{U2}-P_{U4}\right)$$

$$A_F := \frac{D\cdot\left(P_{U1}-P_{U2}\right)}{2}$$

CENTROID

$$C_A := \frac{D+E+F}{2}$$

$$C_B := \frac{2\cdot F}{3}$$

$$C_C := F + \frac{E}{2}$$

$$C_D := F + \frac{2\cdot E}{3}$$

$$C_E := \left(\frac{D}{2}+E+F\right)$$

$$C_F := \left(\frac{2\cdot D}{3}+E+F\right)$$

$$X_U := \frac{A_A\cdot C_A + A_B\cdot C_B + A_C\cdot C_C + A_D\cdot C_D + A_E\cdot C_E + A_F\cdot C_F}{A_A + A_B + A_C + A_D + A_E + A_F} \qquad X_U = 3.32\,\text{ft}$$

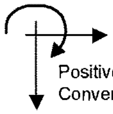
Uplift on Heel $\quad W_U := A_A + A_B + A_C + A_D + A_E + A_F \qquad W_U = 4085\,\frac{\text{lb}}{\text{ft}}$

- HEEL check (cont'd)

Loading:

Bending

$$M_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[W_U \cdot X_U + \frac{P_{BP1} \cdot L_{BP1}}{2} \cdot \frac{L_{BP1}}{3} - \left[\left(W_w + W_s \right) \cdot \left(\frac{D + E + F}{2} \right) + W_H \cdot X_H \right] \right]$$



Positive Sign Convention

$M_H = -4382.16 \cdot \frac{ft \cdot lb}{ft}$ $\quad M_{uH} := |M_H|$

Note:
IF: M_H is < 0
THEN: Steel in top of heel is in tension

Shear

$$V_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \left(W_w + W_s + W_H - W_U - \frac{P_{BP1} \cdot L_{BP1}}{2} \right)$$

$M_{uH} = 4.4 \cdot \frac{kip \cdot ft}{ft}$

$V_H = 953.17 \frac{lb}{ft}$ $\quad V_{uH} := |V_H|$ $\quad V_{uH} = 1 \cdot \frac{kip}{ft}$

Capacity:

Flexural Capacity

$A_{s1} := A_{Heel1}$

$b := 12 \frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 0.58 \cdot in$

$\phi M_{H1} := \phi_B \, A_{s1} \cdot F_y \cdot \left(d_{H1} - \frac{a1}{2} \right)$

$\phi M_{H1} = 21.08 \cdot \frac{kip \cdot ft}{ft}$

$A_{s2} := A_{Heel2}$

$b := 12 \frac{in}{ft}$

$a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a2 = 0.16 \cdot in$

$\phi M_{H2} := \phi_B \, A_{s2} \cdot F_y \cdot \left(d_{H2} - \frac{a2}{2} \right)$

$\phi M_{H2} = 5.85 \cdot \frac{kip \cdot ft}{ft}$

$\phi M_H := if\left(M_H > 0, \phi M_{H2}, \phi M_{H1} \right)$ $\quad \phi M_H = 21.08 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c1} = 19225 \frac{lb}{ft}$

$\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{H2} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c2} = 19389 \frac{lb}{ft}$

$\phi V_H := if\left(M_H > 0, \phi V_{c2}, \phi V_{c1} \right)$ $\quad \phi V_H = 19.23 \cdot \frac{kip}{ft}$

- HEEL check (cont'd)

Factors of Safety

Bending $\phi M_H = 21.08 \cdot \frac{kip \cdot ft}{ft}$ $FS_1 := \frac{\phi M_H}{M_{uH}}$

$M_{uH} = 4.38 \cdot \frac{kip \cdot ft}{ft}$ $FS_1 = 4.81$

$Check1 := if(\phi M_H > 1.5M_{uH}, "OKAY", "NO\ GOOD")$ $Check1 = "OKAY"$

Shear $\phi V_H = 19.23 \cdot \frac{kip}{ft}$ $FS_2 := \frac{\phi V_H}{V_{uH}}$

$V_{uH} = 0.95 \cdot \frac{kip}{ft}$ $FS_2 = 20.17$

$Check2 := if(\phi V_H > 1.5V_{uH}, "OKAY", "NO\ GOOD")$ $Check2 = "OKAY"$

Controlling Factor of Safety

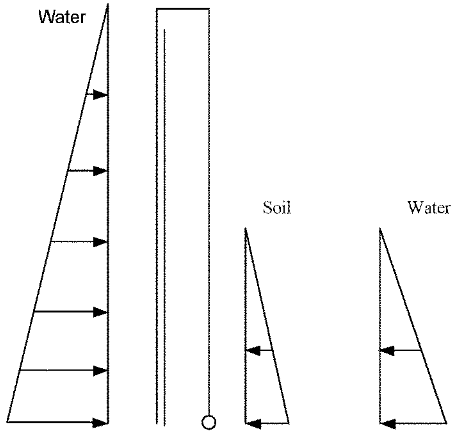
$FS_H := min(FS_1, FS_2)$ $FS_H = 4.81$

Controlling Mechanism

$Comment_H := if(FS_1 > FS_2, "Shear\ in\ Heel", if(M_H > 0, "Flexural\ Bottom\ Steel\ in\ Heel", "Flexural\ Top\ Steel\ in\ Heel"))$

$Comment_H = "Flexural\ Top\ Steel\ in\ Heel"$

- STEM Check



Note:
P_{ss} = Soil pressure present at base of stem
P_{ws2} = Water pressure present at base of stem

$P_{ws1} := (A + B)\cdot\gamma_w$

$P_{ws1} = 593\,\frac{lb}{ft^2}$

$A_{Stem1} := 0.35\,\frac{in^2}{ft}$ $\quad A_{Stem2} := 0.2\,\frac{in^2}{ft}$ $\quad P_{ss} := \frac{J}{I + J}\cdot P_{s1}$ $\quad P_{ws2} := \frac{J}{I + J}\cdot P_{w1}$

$C_{CS1} := 2.8125in$ $\quad C_{CS2} := 2.75in$ $\quad P_{ss} = 106.63\,\frac{lb}{ft^2}$ $\quad P_{ws2} = 375.74\,\frac{lb}{ft^2}$

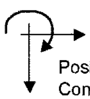
$d_{S1} := G - C_{CS1}$ $\quad d_{S2} := G - C_{CS2}$

Loading:

Bending

$$M_S := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2}\cdot\frac{A + B}{3} - \frac{(P_{ss} + P_{ws2})\cdot(J)}{2}\cdot\frac{J}{3}\right]$$

Note:
IF: M_S is > 0
THEN: Riverside steel is in tension



Positive Sign Convention

$M_S = 7102.8\cdot\frac{ft\cdot lb}{ft}$ $\quad M_{uS} := |M_S|$ $\quad M_{uS} = 7.1\cdot\frac{kip\cdot ft}{ft}$

Shear

$$V_S := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2} - \frac{(P_{ss} + P_{ws2})\cdot(J)}{2}\right]$$

$V_S = 1.67\cdot\frac{kip}{ft}$ $\quad V_{uS} := |V_S|$ $\quad V_{uS} = 1.7\cdot\frac{kip}{ft}$

- STEM Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$ $\qquad$ $A_{s2} := A_{Stem2}$

$b := 12\frac{in}{ft}$ $\qquad$ $b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85f'_c \cdot b}$ $\quad a1 = 0.41 \cdot in$ $\qquad$ $a2 := \frac{A_{s2} \cdot F_y}{0.85f'_c \cdot b}$ $\quad a2 = 0.24 \cdot in$

$\phi M_{S1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{S1} - \frac{a1}{2}\right)$ $\qquad$ $\phi M_{S2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{S2} - \frac{a2}{2}\right)$

$\phi M_{S1} = 16.26 \cdot \frac{kip \cdot ft}{ft}$ $\qquad$ $\phi M_{S2} = 9.38 \cdot \frac{kip \cdot ft}{ft}$

$\phi M_S := if\left(M_S > 0, \phi M_{S1}, \phi M_{S2}\right)$ $\qquad$ $\phi M_S = 16.26 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{S1} \cdot \sqrt{f'_c \cdot psi}$ $\qquad$ $\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{S2} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c1} = 20622 \frac{lb}{ft}$ $\qquad$ $\phi V_{c2} = 20704 \frac{lb}{ft}$

$\phi V_S := if\left(M_S > 0, \phi V_{c1}, \phi V_{c2}\right)$ $\qquad$ $\phi V_S = 20.62 \cdot \frac{kip}{ft}$

Factors of Safety

Bending

$\phi M_S = 16.26 \cdot \frac{kip \cdot ft}{ft}$ $\qquad$ $FS_3 := \frac{\phi M_S}{M_{uS}}$

$M_{uS} = 7.1 \cdot \frac{kip \cdot ft}{ft}$ $\qquad$ $FS_3 = 2.29$

$Check3 := if\left(\phi M_S > 1.5M_{uS}, \text{"OKAY"}, \text{"NO GOOD"}\right)$ $\qquad$ $Check3 = \text{"OKAY"}$

Shear

$\phi V_S = 20.62 \cdot \frac{kip}{ft}$ $\qquad$ $FS_4 := \frac{\phi V_S}{V_{uS}}$

$V_{uS} = 1.67 \cdot \frac{kip}{ft}$ $\qquad$ $FS_4 = 12.35$

$Check4 := if\left(\phi V_S > 1.5V_{uS}, \text{"OKAY"}, \text{"NO GOOD"}\right)$ $\qquad$ $Check4 = \text{"OKAY"}$

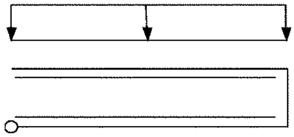
Controlling Factor of Safety

$FS_S := min\left(FS_3, FS_4\right)$ $\qquad$ $FS_S = 2.29$

Controlling Mechanism

- TOE Check

Soil



Slab

$W_s := \gamma \cdot H \cdot (J)$ $\quad W_s = 1911.87\,\frac{lb}{ft}$

$A_{Toe1} := 0.29\,\frac{in^2}{ft}$ $\quad C_{CT1} := 3.375in$ $\quad d_{T1} := I - C_{CT1}$

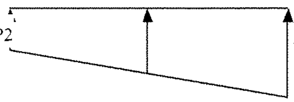
$A_{Toe2} := 0.133\,\frac{in^2}{ft}$ $\quad C_{CT2} := 3.25in$ $\quad d_{T1} = 14.63 \cdot in$

$L_{BP2} := \mathrm{if}(H \le L_{BP}, H, L_{BP})$

$L_{BP2} = 3.5\,ft$

$d_{T2} := I - C_{CT2}$

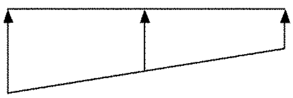
Bearing Pressure P_{BP2}



$P_{BP} = 853.9\,\frac{lb}{ft^2}$

$d_{T2} = 14.75 \cdot in$

$$P_{BP2} := \mathrm{if}\left[L_{BP} < (D + E + F + G + H), P_{BP} - P_{BP} \cdot \frac{L_{BP2}}{L_{BP}}, P_{BP} - L_{BP2} \cdot \left(\frac{P_{BP} - P_{BPa}}{L_{BP}}\right)\right] = 591.97\,\frac{lb}{ft^2}$$



$P_{U6} = 494.4\,\frac{lb}{ft^2}$

Uplift

$$P_{U5} := \left[\left[(P_{U3} - P_{U6}) \cdot \frac{H}{F + G + H} + P_{U6}\right]\right]$$

$$P_{U5} = 552.83\,\frac{lb}{ft^2}$$

Bearing Pressure

$$L_{bp} := \frac{(L_{BP2} \cdot P_{BP2}) \cdot \left(\frac{L_{BP2}}{2}\right) + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2} \cdot \left(\frac{2L_{BP}}{3}\right)}{L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2}} \qquad L_{bp} = 2.81\,ft$$

$$W_{bp} := L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2} \qquad W_{bp} = 2530\,\frac{lb}{ft}$$

Uplift

$$L_u := \frac{H \cdot P_{U6} \cdot \frac{H}{2} + \frac{(P_{U5} - P_{U6}) \cdot H}{2} \cdot \frac{H}{3}}{H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2}} \qquad L_u = 1.72\,ft$$

$$W_u := H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2} \qquad W_u = 1832.66\,\frac{lb}{ft}$$

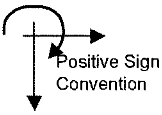
Toe Check (continued)

Loading:

Bending

$$M_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[\left[W_s + (H \cdot I) \cdot \gamma_c \right] \cdot \frac{H}{2} - \left(W_{bp} \cdot L_{bp} + W_u \cdot L_u \right) \right]$$

Note:
IF: $M_T < 0$
THEN: Bottom steel is in tension



$$M_T = -5.5 \cdot \frac{kip \cdot ft}{ft} \qquad M_{uT} := |M_T| \qquad M_{uT} = 5.54 \cdot \frac{kip \cdot ft}{ft}$$

Shear

$$V_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[W_s + (H \cdot I) \cdot \gamma_c - W_{bp} - W_u \right]$$

$$V_T = -1.66 \cdot \frac{kip}{ft} \qquad V_{uT} := |V_T| \qquad V_{uT} = 1.66 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

$$A_{s1} := A_{Toe1} \qquad A_{s2} := A_{Toe2}$$

$$b := 12 \frac{in}{ft} \qquad b := 12 \frac{in}{ft}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 0.34 \cdot in \qquad a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \qquad a2 = 0.16 \cdot in$$

$$\phi M_{T1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{T1} - \frac{a1}{2} \right) \qquad \phi M_{T2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{T2} - \frac{a2}{2} \right)$$

$$\phi M_{T1} = 12.58 \cdot \frac{kip \cdot ft}{ft} \qquad \phi M_{T2} = 5.85 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_T := \mathrm{if}\left(M_T > 0, \phi M_{T1}, \phi M_{T2} \right) \qquad \phi M_T = 5.85 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{T1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{T2} \cdot \sqrt{f'_c \cdot psi}$$

$$\phi V_{c1} = 19225 \frac{lb}{ft} \qquad \phi V_{c2} = 19389 \frac{lb}{ft}$$

$$\phi V_T := \mathrm{if}\left(M_{uT} > 0, \phi V_{c1}, \phi V_{c2} \right) \qquad \phi V_T = 19.23 \cdot \frac{kip}{ft}$$

Toe Check (continued)

Factors of Safety

Bending

$\phi M_T = 5.85 \cdot \frac{kip \cdot ft}{ft}$

$FS_5 := \frac{\phi M_T}{M_{uT}}$

$M_{uT} = 5.54 \cdot \frac{kip \cdot ft}{ft}$

$FS_5 = 1.06$

$Check5 := if(\phi M_T > 1.5 \cdot M_{uT}, \text{"OKAY"}, \text{"NO GOOD"})$

$Check5 = \text{"NO GOOD"}$

Shear

$\phi V_T = 19.23 \cdot \frac{kip}{ft}$

$FS_6 := \frac{\phi V_T}{V_{uT}}$

$V_{uT} = 1.66 \cdot \frac{kip}{ft}$

$FS_6 = 11.56$

$Check6 := if(\phi V_T > 1.5 V_{uT}, \text{"OKAY"}, \text{"NO GOOD"})$

$Check6 = \text{"OKAY"}$

Controlling Factor of Safety

$FS_T := min(FS_5, FS_6)$

$FS_T = 1.06$

Controlling Mechanism

$Comment_T := if(FS_5 > FS_6, \text{"Shear in Heel"}, if(M_T > 0, \text{"Flexural Top Steel in Toe"}, \text{"Flexural Bottom Steel in Toe"}))$

$Comment_T = \text{"Flexural Bottom Steel in Toe"}$

Overall Factor of Safety (without considering multiple layers of steel)

Factor of Safety

$$FoS := \min(FS_H, FS_S, FS_T)$$

Limiting Mechanism

$$Mechanism := Comment_H$$

$$Mechanism := if(FoS = FS_S, Comment_S, Mechanism)$$

$$Mechanism := if(FoS = FS_T, Comment_T, Mechanism)$$

$$FoS = 1.06$$

Mechanism = "Flexural Bottom Steel in Toe"

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis. If the factor of safety is lower than 1.5 a probablility analysis is required.

US Army Corps of Engineers

Probability of Failure
CIDMO 12ft floodwall spread footing Bending in Toe

Comp by:KSM 8-2-11
Chkd by:

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. This structure does not meet the strength 1.5 factor of safety for which it has been determined 99.8% reliability can be assigned. See mathcad analysis of the wall for existing condition strength check.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength
3. Material Properties used:
   Mean Concrete Strength, $f'_{cM} := 3750 \cdot psi$
   Mean Steel Strength, $F_{yM} := 45 \cdot ksi$
4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. *Calculating Factors of Safety*

WATER AT TOP OF WALL

Condition under consideration from strength check: Mu for toe (from mathcad strength analysis).

$M_u := 6.75 \frac{kip \cdot ft}{ft}$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$A_s := 0.133 \frac{in^2}{ft}$ Area of Steel

$dist_{CL} := 3.25in$ Distance from bottom of footing to centerline of reinforcement

$T_{toe} := 1.5ft$ Thickness of footing at toe/stem interface

$b := 12 \frac{in}{ft}$ 1 ft strip of wall analyzed

$d := T_{toe} - dist_{CL}$ $\quad d = 14.75 \cdot in$

Mean Concrete Strength and Steel Yield Strength

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cM} \cdot b}$ $\quad a = 0.156 \cdot in$

$\phi M_M := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_M = 7.32 \cdot \frac{kip \cdot ft}{ft}$

$FS_D := \frac{\phi M_M}{M_u}$ $\quad FS_D = 1.084$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM} \cdot 0.14$ $\quad f'_{cU} = 4275 \cdot psi$

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cU} \cdot b}$ $\quad a = 0.137 \cdot in$

$\phi M_{cU} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{cU} = 7.32 \cdot \frac{kip \cdot ft}{ft}$

$FS_{fcu} := \frac{\phi M_{cU}}{M_u}$ $\quad FS_{fcu} = 1.085$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14 \qquad f'_{cL} = 3225 \cdot psi$

$$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cL} \cdot b} \qquad a = 0.182 \cdot in$$

$$\phi M_{cL} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right) \qquad \phi M_{cL} = 7.31 \cdot \frac{kip \cdot ft}{ft}$$

$$FS_{fcl} := \frac{\phi M_{cL}}{M_u} \qquad FS_{fcl} = 1.083$$

Upper Steel Yield Strength

$F_{yU} := F_{yM} + F_{yM} \cdot 0.14 \qquad F_{yU} = 51.3 \cdot ksi$

$$a := \frac{A_s \cdot F_{yU}}{0.85 f'_{cM} \cdot b} \qquad a = 0.178 \cdot in$$

$$\phi M_{sU} := \phi_B A_s \cdot F_{yU} \cdot \left(d - \frac{a}{2}\right) \qquad \phi M_{sU} = 8.34 \cdot \frac{kip \cdot ft}{ft}$$

$$FS_{Fyu} := \frac{\phi M_{sU}}{M_u} \qquad FS_{Fyu} = 1.235$$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM} \cdot 0.14 \qquad F_{yL} = 38.7 \cdot ksi$

$$a := \frac{A_s \cdot F_{yL}}{0.85 f'_{cM} \cdot b} \qquad a = 0.135 \cdot in$$

$$\phi M_{sL} := \phi_B A_s \cdot F_{yL} \cdot \left(d - \frac{a}{2}\right) \qquad \phi M_{sL} = 6.3 \cdot \frac{kip \cdot ft}{ft}$$

$$FS_{Fyl} := \frac{\phi M_{sL}}{M_u} \qquad FS_{Fyl} = 0.933$$

$FS_D = 1.084 \qquad FS_{fcu} = 1.085 \qquad FS_{Fyu} = 1.235$

$FS_{fcl} = 1.083 \qquad FS_{Fyl} = 0.933$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{Fyu} - FS_{Fyl}$ $\qquad$ $\Delta F_{Fy} = 0.302$

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\qquad$ $\Delta F_{fc} = 1.651 \times 10^{-3}$

ACI EQ (11-4)

$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2} \qquad \sigma_F = 0.151$$

$$V_F := \frac{\sigma_F}{FS_D} \qquad V_F = 0.139$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = 0.513$$

$$R := \text{cnorm}\left(\beta_{LN}\right) \qquad R = 69.61 \cdot \%$$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$$P_F := 1 - R \qquad P_F = 30.39 \cdot \%$$

WATER AT 1ft down from top of WALL

Condition under consideration from strength check: Mu for toe (from mathcad strength analysis).

$M_u := 5.54 \frac{kip \cdot ft}{ft}$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$A_s := 0.133 \frac{in^2}{ft}$ Area of Steel

$dist_{CL} := 3.25in$ Distance from bottom of footing to centerline of reinforcement

$T_{toe} := 1.5ft$ Thickness of footing at toe/stem interface

$b := 12 \frac{in}{ft}$ 1 ft strip of wall analyzed

$d := T_{toe} - dist_{CL}$ $\quad d = 14.75 \cdot in$

Mean Concrete Strength and Steel Yield Strength

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cM} \cdot b}$ $\quad a = 0.156 \cdot in$

$\phi M_M := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_M = 7.32 \cdot \frac{kip \cdot ft}{ft}$

$FS_D := \frac{\phi M_M}{M_u}$ $\quad FS_D = 1.321$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f_{cU} := f_{cM} + f_{cM} \cdot 0.14$ $\quad f_{cU} = 4275 \cdot psi$

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cU} \cdot b}$ $\quad a = 0.137 \cdot in$

$\phi M_{cU} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{cU} = 7.32 \cdot \frac{kip \cdot ft}{ft}$

$FS_{fcu} := \frac{\phi M_{cU}}{M_u}$ $\quad FS_{fcu} = 1.322$

Probablity of Failure Sheet 5 of 7

Lower Concrete Strength

$f_{cL} := f_{cM} - f_{cM} \cdot 0.14$ $\quad f_{cL} = 3225 \cdot psi$

$a := \dfrac{A_s \cdot F_{yM}}{0.85 f_{cL} \cdot b}$ $\quad a = 0.182 \cdot in$

$\phi M_{cL} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \dfrac{a}{2}\right)$ $\quad \phi M_{cL} = 7.31 \cdot \dfrac{kip \cdot ft}{ft}$

$FS_{fcl} := \dfrac{\phi M_{cL}}{M_u}$ $\quad FS_{fcl} = 1.32$

Upper Steel Yield Strength

$F_{yU} := F_{yM} + F_{yM} \cdot 0.14$ $\quad F_{yU} = 51.3 \cdot ksi$

$a := \dfrac{A_s \cdot F_{yU}}{0.85 f_{cM} \cdot b}$ $\quad a = 0.178 \cdot in$

$\phi M_{sU} := \phi_B A_s \cdot F_{yU} \cdot \left(d - \dfrac{a}{2}\right)$ $\quad \phi M_{sU} = 8.34 \cdot \dfrac{kip \cdot ft}{ft}$

$FS_{Fyu} := \dfrac{\phi M_{sU}}{M_u}$ $\quad FS_{Fyu} = 1.505$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM} \cdot 0.14$ $\quad F_{yL} = 38.7 \cdot ksi$

$a := \dfrac{A_s \cdot F_{yL}}{0.85 f_{cM} \cdot b}$ $\quad a = 0.135 \cdot in$

$\phi M_{sL} := \phi_B A_s \cdot F_{yL} \cdot \left(d - \dfrac{a}{2}\right)$ $\quad \phi M_{sL} = 6.3 \cdot \dfrac{kip \cdot ft}{ft}$

$FS_{Fyl} := \dfrac{\phi M_{sL}}{M_u}$ $\quad FS_{Fyl} = 1.137$

$FS_D = 1.321$ $\quad FS_{fcu} = 1.322$ $\quad FS_{Fyu} = 1.505$

$FS_{fcl} = 1.32$ $\quad FS_{Fyl} = 1.137$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{Fyu} - FS_{Fyl}$ $\qquad \Delta F_{Fy} = 0.368$

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\qquad \Delta F_{fc} = 2.012 \times 10^{-3}$

ACI EQ (11-4)

$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2} \qquad \sigma_F = 0.184$$

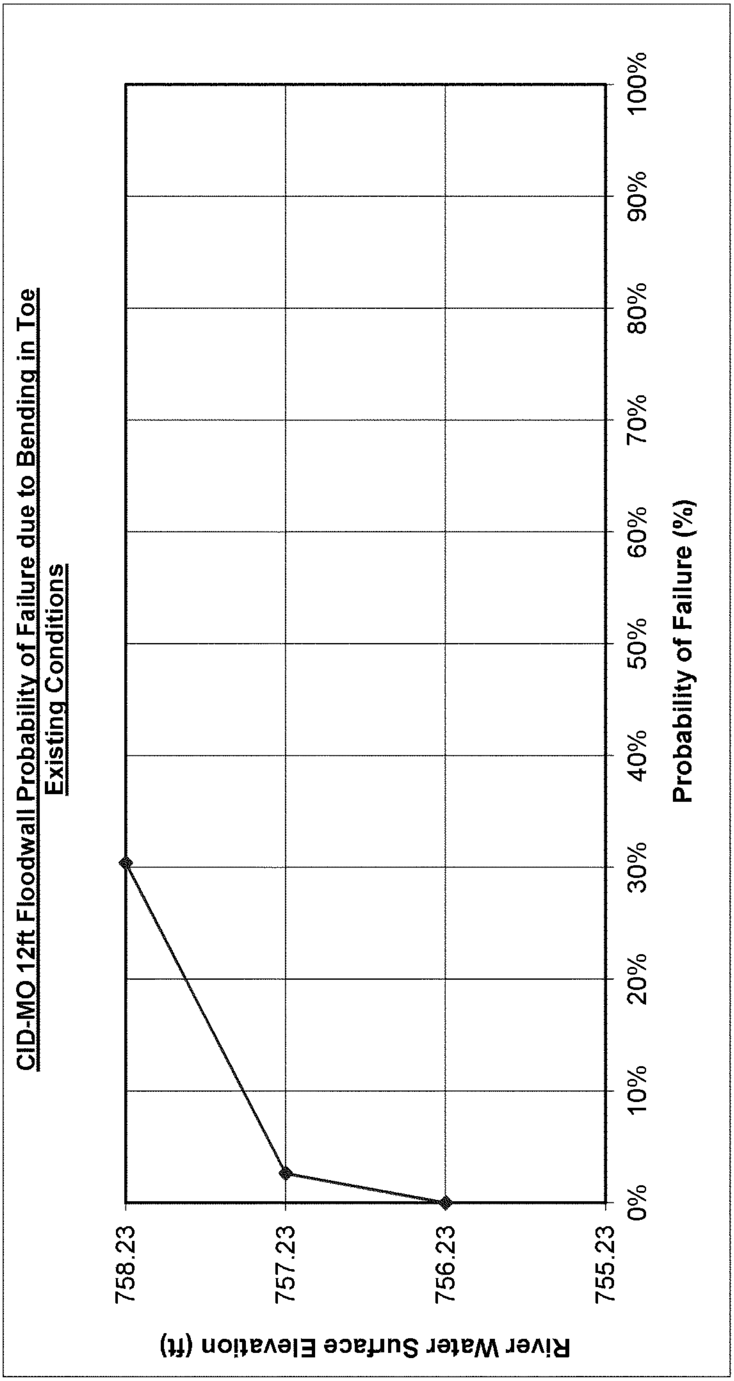
$$V_F := \frac{\sigma_F}{FS_D} \qquad V_F = 0.139$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = 1.939$$

$R := \mathrm{cnorm}(\beta_{LN})$ $\qquad R = 97.37 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad P_F = 2.63 \cdot \%$



CID-MO 12ft Floodwall Probability of Failure due to Bending in Toe
Existing Conditions
River Water Surface Elevation (ft)
758.23
757.23
756.23
755.23
0%
10%
20%
30%
40%
50%
60%
70%
80%
90%
100%
Probability of Failure (%)

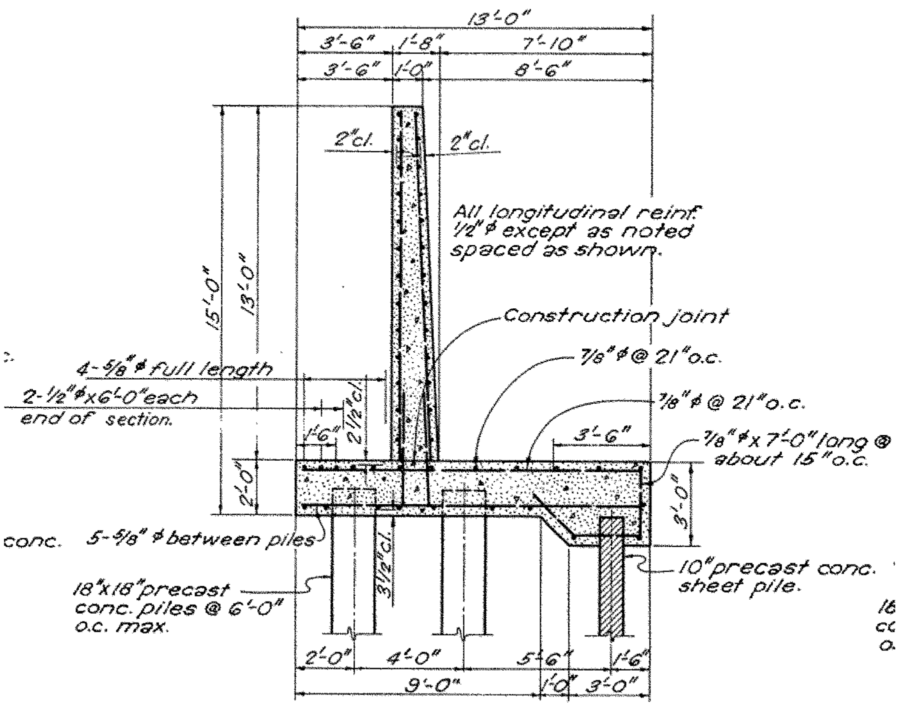
EXHIBIT A-8.3

Pile Founded Floodwall Sample Calculations

US Army Corps of Engineers

CID-MO Floodwall
Floodwall Analysis - Existing Conditions
Type "R" floodwall
Kansas City Levees

Comp by: KSM 12-6-11
Chkd by:



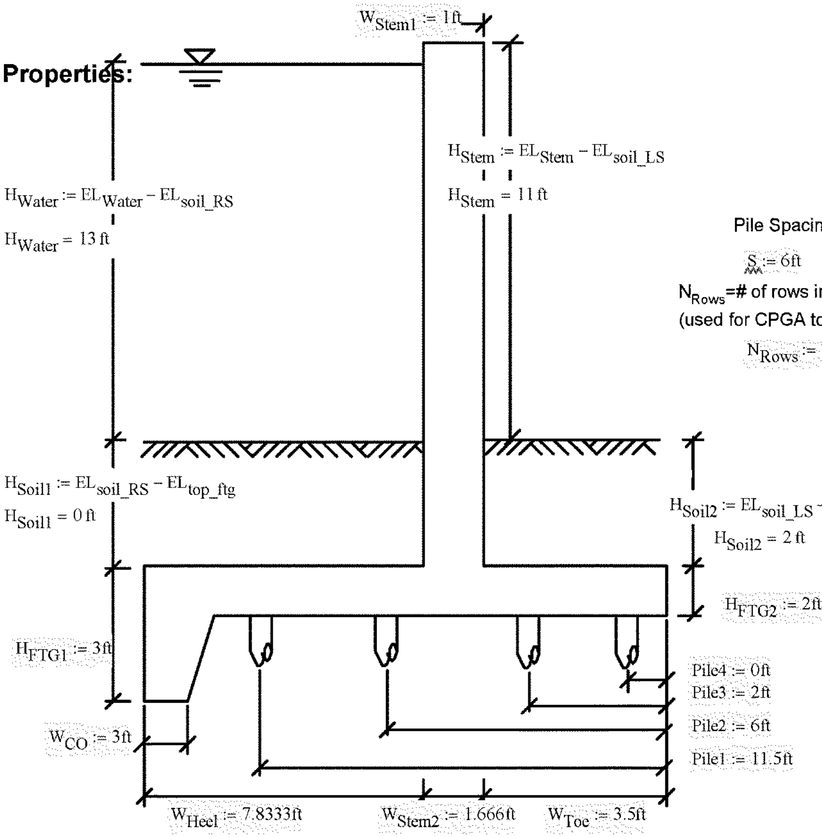
Typical Section from 1946 Record Drawings

Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $ksf \equiv \frac{1000lb}{ft^2}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$

$EL_{Stem} := 100ft$ $EL_{Water} := 100ft$ $EL_{soil\_RS} := 87ft$ $EL_{soil\_LS} := 89ft$ $EL_{top\_ftg} := 87ft$

Properties:



Pile Spacing

$S := 6ft$

N_{Rows}=# of rows in monolith (used for CPGA total loads)

$N_{Rows} := 7$

Concrete Properties $\gamma_c := 150pcf$ Soil Properties $\gamma := 115pcf$ $\phi := 22deg$

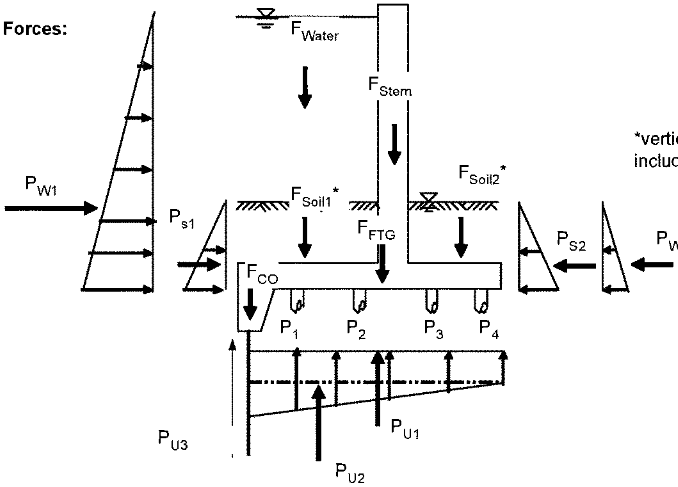
Water Weight $\gamma_w := 62.4pcf$ $K_u := 1 - \sin(\phi)$ $K_u = 0.63$

Lengths:

$L_{CO} := H_{FTG1} - H_{FTG2}$ $\quad L_{W1} := H_{Water} + H_{Soil1} + H_{FTG1}$ $\quad L_{STM} := H_{Stem} + H_{Soil2}$

$L_{FTG} := W_{Heel} + W_{Stem2} + W_{Toe}$ $\quad L_{PU1} := H_{Soil2} + H_{FTG2}$

Forces:



$L_{W1} = 16\,\text{ft}$

$L_{PU1} = 4\,\text{ft}$

*vertical soil loads include water

$$F_{Stem} := \left(\frac{W_{Stem1} + W_{Stem2}}{2}\right) \cdot L_{STM} \cdot \gamma_c \qquad F_{Stem} = 2599.35\,\frac{\text{lb}}{\text{ft}}$$

$$F_{FTG} := H_{FTG2} \cdot L_{FTG} \cdot \gamma_c \qquad F_{FTG} = 3899.79\,\frac{\text{lb}}{\text{ft}}$$

$$F_{CO} := L_{CO} \cdot W_{CO} \cdot \gamma_c \qquad F_{CO} = 450\,\frac{\text{lb}}{\text{ft}}$$

$$F_{Soil1} := H_{Soil1} \cdot W_{Heel} \cdot \gamma \qquad F_{Soil1} = 0\,\frac{\text{lb}}{\text{ft}}$$

$$F_{Soil2} := H_{Soil2} \cdot W_{Toe} \cdot \gamma \qquad F_{Soil2} = 805\,\frac{\text{lb}}{\text{ft}}$$

$$F_{Water1} := H_{Water} \cdot W_{Heel} \cdot \gamma_w \qquad F_{Water1} = 6354.37\,\frac{\text{lb}}{\text{ft}}$$

$$P_{W1} := \frac{1}{2} \cdot L_{W1}^{\;2} \cdot \gamma_w \qquad P_{W1} = 7987.2\,\frac{\text{lb}}{\text{ft}}$$

$$P_{W2} := -\left[\frac{1}{2} \cdot \left(H_{FTG1} + H_{Soil2}\right)^2 \cdot \gamma_w\right] \qquad P_{W2} = -780\,\frac{\text{lb}}{\text{ft}}$$

$$P_{S1} := K_u \cdot \left[\frac{1}{2} \cdot \left(H_{Soil1} + H_{FTG1}\right)^2 \cdot \left(\gamma - \gamma_w\right)\right] \qquad P_{S1} = 148.03\,\frac{\text{lb}}{\text{ft}}$$

$$P_{S2} := -K_u \cdot \left[\frac{1}{2} \cdot \left(H_{Soil2} + H_{FTG1}\right)^2 \cdot \left(\gamma - \gamma_w\right)\right] \qquad P_{S2} = -411.2\,\frac{\text{lb}}{\text{ft}}$$

Forces Con't

$$p_{U3} := (L_{W1})\cdot\gamma_w = 998.4\frac{lb}{ft^2}$$ <---UA

$$P_{U3} := \frac{-W_{CO}}{2}\cdot p_{U3} = -1497.6\frac{lb}{ft}$$

$$p_{U2} := -\left[L_{W1}\cdot\gamma_w - 0.5\cdot\left(L_{W1}\cdot\gamma_w - \gamma_w\cdot L_{PU1}\right)\right] = -624\frac{lb}{ft^2}$$ <---UA'

$$p_{U1} := -\left[\gamma_w\cdot L_{PU1} + \frac{L_{PU1}}{L_{FTG} - \frac{W_{CO}}{2} + L_{PU1}}\left(\left|p_{U2}\right| - \gamma_w\cdot L_{PU1}\right)\right] = -346.22\frac{lb}{ft^2}$$ <---UB

$$P_{U1} := p_{U1}\left(L_{FTG} - \frac{W_{CO}}{2}\right) = -3981.33\frac{lb}{ft}$$

$$P_{U2i} := p_{U2} - p_{U1} = -277.78\frac{lb}{ft^2}$$

$$P_{U2} := 0.5\cdot\left(P_{U2i}\right)\cdot\left(L_{FTG} - \frac{W_{CO}}{2}\right) = -1597.12\frac{lb}{ft}$$

$$P_{U2} = -1597.12\frac{lb}{ft}$$

$$P_{U1} = -3981.33\frac{lb}{ft}$$

$$P_{U3} = -1497.6\frac{lb}{ft}$$

Inputs for CPGA

Sum moments around bottom corner of Toe:

Moment Arm
from bottom corner of Toe

$$Arm_{Stem} := -\left(W_{Toe} + \frac{W_{Stem2}}{2}\right)$$

$$Arm_{FTG} := -\frac{L_{FTG}}{2}$$

$$Arm_{CO} := -L_{FTG} + \frac{W_{CO}}{2}$$

$$Arm_{Soil1} := -\left(W_{Toe} + W_{Stem2} + \frac{W_{Heel}}{2}\right)$$

$$Arm_{Soil2} := -\frac{W_{Toe}}{2}$$

$$Arm_{Water1} := Arm_{Soil1}$$

$$Arm_{W1} := \frac{1}{3} \cdot L_{W1} - L_{CO}$$

$$Arm_{W2} := \frac{1}{3} \cdot \left(L_{PU1} + L_{CO}\right) - L_{CO}$$

$$Arm_{S1} := \frac{1}{3} \cdot \left(H_{Soil1} + H_{FTG1}\right) - L_{CO}$$

$$Arm_{S2} := Arm_{W2}$$

$$Arm_{U1} := -\frac{L_{FTG} - \frac{W_{CO}}{2}}{2}$$

$$Arm_{U2} := -\frac{2\left(L_{FTG} - \frac{W_{CO}}{2}\right)}{3}$$

$$Arm_{U3} := -\left(L_{FTG} - \frac{W_{CO}}{4}\right)$$

$Arm_{Stem} = -4.33\,ft$

$Arm_{FTG} = -6.5\,ft$

$Arm_{CO} = -11.5\,ft$

$Arm_{Soil1} = -9.08\,ft$

$Arm_{Soil2} = -1.75\,ft$

$Arm_{Water1} = -9.08\,ft$

$Arm_{W1} = 4.33\,ft$

$Arm_{W2} = 0.67\,ft$

$Arm_{S1} = 0\,ft$

$Arm_{S2} = 0.67\,ft$

$Arm_{U1} = -5.75\,ft$

$Arm_{U2} = -7.67\,ft$

$Arm_{U3} = -12.25\,ft$

Moment (about bottom corner of toe)

$M_{Stem} := F_{Stem} \cdot Arm_{Stem}$	$M_{Stem} = -11.26 \cdot \frac{kip \cdot ft}{ft}$
$M_{FTG} := F_{FTG} \cdot Arm_{FTG}$	$M_{FTG} = -25.35 \cdot \frac{kip \cdot ft}{ft}$
$M_{CO} := F_{CO} \cdot Arm_{CO}$	$M_{CO} = -5.17 \cdot \frac{kip \cdot ft}{ft}$
$M_{Soil1} := F_{Soil1} \cdot Arm_{Soil1}$	$M_{Soil1} = 0 \cdot \frac{kip \cdot ft}{ft}$
$M_{Soil2} := F_{Soil2} \cdot Arm_{Soil2}$	$M_{Soil2} = -1.41 \cdot \frac{kip \cdot ft}{ft}$
$M_{Water1} := F_{Water1} \cdot Arm_{Water1}$	$M_{Water1} = -57.71 \cdot \frac{kip \cdot ft}{ft}$
$M_{W1} := P_{W1} \cdot Arm_{W1}$	$M_{W1} = 34.61 \cdot \frac{kip \cdot ft}{ft}$
$M_{W2} := P_{W2} \cdot Arm_{W2}$	$M_{W2} = -0.52 \cdot \frac{kip \cdot ft}{ft}$
$M_{S1} := P_{S1} \cdot Arm_{S1}$	$M_{S1} = 0 \cdot \frac{kip \cdot ft}{ft}$
$M_{S2} := P_{S2} \cdot Arm_{S2}$	$M_{S2} = -0.27 \cdot \frac{kip \cdot ft}{ft}$
$M_{U1} := P_{U1} \cdot Arm_{U1}$	$M_{U1} = 22.89 \cdot \frac{kip \cdot ft}{ft}$
$M_{U2} := P_{U2} \cdot Arm_{U2}$	$M_{U2} = 12.24 \cdot \frac{kip \cdot ft}{ft}$
$M_{U3} := P_{U3} \cdot Arm_{U3}$	$M_{U3} = 18.34 \cdot \frac{kip \cdot ft}{ft}$

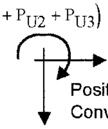
$$\Sigma M := \left(M_{Stem} + M_{FTG} + M_{CO} + M_{Soil1} + M_{Soil2} + M_{Water1} + M_{W1} + M_{W2} + M_{S1} + M_{S2} + M_{U1} + M_{U2} + M_{U3}\right)$$

$$\Sigma M = -13.61 \cdot \frac{ft \cdot kip}{ft}$$

$$\Sigma Vertical := \left(F_{Stem} + F_{FTG} + F_{CO} + F_{Soil1} + F_{Soil2} + F_{Water1} + P_{U1} + P_{U2} + P_{U3}\right) \qquad \Sigma Vertical = 7.03 \cdot \frac{kip}{ft}$$

$$\Sigma Horizontal := \left(P_{W1} + P_{W2} + P_{S1} + P_{S2}\right) \qquad \Sigma Horizontal = 6.94 \cdot \frac{kip}{ft}$$

Loading about the stem.



Positive Sign Convention

Find Pile Loads Using CPGA:

Number := if(Pile5 = 0, 4, 5)
Number := if(Pile4 = 0, 3, Number)
Number := if(Pile3 = 0, 2, Number)
Number := if(Pile2 = 0, 1, Number)
Number := if(Pile1 = 0, 0, Number)

Total number of piles:

Number = 3

Concrete Pile Properties

PILES 1 THRU 14 IN CPGA

Dimension of square pile, $D := 15in$

Assume all piles to pile cap connections are fixed (details show rebar extending into base).

Note - the pile tapers from 18" to 12" over 21feet. Used a dimension at half way down the pile for analysis purposes.

Pile Properties:

$fc := 4000\ psi \qquad E_C := 57 \cdot \sqrt{fc} \qquad E_C = 3605\ ksi$

$I := \frac{D}{12} \qquad I = 4218.75 \cdot in^4 \qquad Area := D^2 \qquad Area = 225 \cdot in^2$

C33=2.0

B66 = 0 torsional stiffness

Length = 20ft

Design Load Strengths:

AC = Allowable Pile Compression Load, = 37.17 k

AT = Allowable Pile Tension Load, =20.25 k

The following design strengths are from the existing pile capacity MathCAD file:

PO = Axial Compression Design Load Strength, 752 k

PT = Axial Tension Design Load Strength, Fy*As= 192 k

PB = Axial Design Load Strength at Balanced Condition, 362 k

MB = Design Moment Strength at Balanced Condition, 2057 k-in

MO = Design Moment Strength Under Pure Flexure, 1089 k-in

Soil:

NH

Soil Modulus, 0.06 k/in^3 (See Table Below)

PILES 15 THRU 56 IN CPGA

Concrete Sheet Pile Properties

Dimension of sheet pile, $D_1 := 12in$, $D_2 := 10in$

Note - D1 is longitudinal and D2 is transverse to the wall stem

Pile Spacing, S=1ft

Assume all sheet pile to pile cap connections are fixed (details show rebar extending into base).

Pile Properties:

$fc := 4000\ psi$ $\quad E_C := 57 \cdot \sqrt{fc}$ $\quad E_C = 3605\ ksi$

$$I_1 := \frac{D_1 \cdot D_2^3}{12} \qquad I_2 := \frac{D_2 \cdot D_1^3}{12}$$

$$I_1 = 1000 \cdot in^4 \qquad I_2 = 1440 \cdot in^4$$

$Area := D_1 \cdot D_2$ $\quad Area = 120 \cdot in^2$

BOO = 0 torsional stiffness

Length = 15ft

Design Load Strengths:

AC = Allowable Pile Compression Load, = 13.43 k/lin.ft.

AT = Allowable Pile Tension Load, =10.10 k/lin.ft.

The following design strengths are from the existing pile capacity MathCAD file:

PO = Axial Compression Design Load Strength, 378 k

PT = Axial Tension Design Load Strength, Fy*As=70 k

PB = Axial Design Load Strength at Balanced Condition, 163 k

MB = Design Moment Strength at Balanced Condition, 631 k-in

MO = Design Moment Strength Under Pure Flexure, 290 k-in

Soil:

NH

Soil Modulus, 0.06 k/in3 (See Table Below)

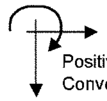
Soil Modulus Parameter k for Clays			
Average Undrained Shear Strength		Static	Cyclic
Soft Clay	c = 1.74 to 3.47 psi 250 to 500 psf 12 to 24 KPa	30 pci 8,140 KPa/m	-- --
Medium Clay	c = 3.47 to 6.94 psi 500 to 1000 psf 24 to 48 KPa	100 pci 27,150 KPa/m	-- --
Stiff Clay	c = 6.94 to 13.9 psi 1000 to 2000 psf 48 to 96 KPa	500 pci 136,000 KPa/m	200 pci 54,300 KPa/m
Very Stiff Clay	c = 13.9 to 27.8 psi 2000 to 4000 psf 96 to 192 KPa	1000 pci 271,000 KPa/m	400 pci 108,500 KPa/m
Hard Clay	c = 27.8 to 55.6 psi 4000 to 8000 psf 192 to 383 KPa	2000 pci 543,000 KPa/m	800 pci 217,000 KPa/m

Use 60 pci for k.

$\Sigma M_T := \Sigma M \cdot S \cdot N_{Rows}$

$\Sigma Vertical_T := \Sigma Vertical \cdot S \cdot N_{Rows}$

$\Sigma Horizontal_T := \Sigma Horizontal \cdot S \cdot N_{Rows}$



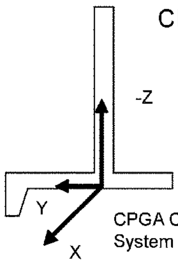
Positive Sign Convention

$\Sigma M_T = -571.68 \cdot ft \cdot kip$

$\Sigma Vertical_T = 295.36 \cdot kip$

$\Sigma Horizontal_T = 291.65 \cdot kip$

CPGA LOADING FOR WATER AT TOP OF WALL



CPGA Coordinate System

Applied Loads

$PY := -\Sigma Horizontal_T$

$PZ := \Sigma Vertical_T$

$MX := -\Sigma M_T$

- Signs added to agree with CPGA sign convention.

$PY = -291.65 \cdot kip$

$PZ = 295.36 \cdot kip$

$MX = 571.68 \cdot ft \cdot kip$

```
CPGA - CASE PILE GROUP ANALYSIS PROGRAM
RUN DATE: 07-DEC-2011    RUN TIME: 14.42.38


     FOR PILES WITH UNSUPPORTED HEIGHT:
           A. CPGA CANNOT CALCULATE PMAXMOM FOR NH TYPE SOIL
           B. THE ALLOWABLE STRESS CHECKS, ASC AND AST, ARE
              NOT FULLY DEVELOPED FOR UNSUPPORTED PILES.
              WORK IS IN PROGRESS TO COMPLETE THIS ASPECT OF CPGA.

     ELASTIC CENTER LOCATION IS NOT COMPUTED FOR 3-DIMENSIONAL PROBLEMS.


CID-MO SECTION TYPE R
DATA UNKNOWN - REJECTED.


THERE ARE  56 PILES AND
            1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                        X            Y           Z
                                      -----        -----       -----
WITH DIAGONAL COORDINATES = (       -20.50 ,      2.00 ,        .00 )
                            (        20.50 ,     11.50 ,        .00 )

*******************************************************************************

           PILE PROPERTIES AS INPUT


        E             I1             I2             A            C33           B66
       KSI           IN**4          IN**4          IN**2
   .36050E+04    .42180E+04    .42180E+04    .22500E+03    .20000E+01    .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

    1    2    3    4    5    6    7    8    9   10   11   12   13   14


        E             I1             I2             A            C33           B66
       KSI           IN**4          IN**4          IN**2
   .36050E+04    .10000E+04    .14400E+04    .12000E+03    .20000E+01    .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
   31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
   47   48   49   50   51   52   53   54   55   56

*******************************************************************************

           SOIL DESCRIPTIONS AS INPUT


    NH      ESOIL       LENGTH         L             LU
           K/IN**3                     FT            FT
           .60000E-01     L        .20000E+02    .00000E+00

  ESOIL(ORIGINAL)      RGROUP      RCYCLIC
   K/IN**3
   .60000E-01         .1000E+01   .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

    1    2    3    4    5    6    7    8    9   10   11   12   13   14


    NH      ESOIL       LENGTH         L             LU
           K/IN**3                     FT            FT
           .60000E-01     L        .15000E+02    .00000E+00

  ESOIL(ORIGINAL)      RGROUP      RCYCLIC
   K/IN**3
   .60000E-01         .1000E+01   .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
   31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
   47   48   49   50   51   52   53   54   55   56

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PILE STIFFNESSES AS CALCULATED FROM PROPERTIES

.14829E+03	.00000E+00	.00000E+00	.00000E+00	.66141E+04	.00000E+00
.00000E+00	.14829E+03	.00000E+00	-.66141E+04	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.67594E+04	.00000E+00	.00000E+00	.00000E+00
.00000E+00	-.66141E+04	.00000E+00	.47570E+06	.00000E+00	.00000E+00
.66141E+04	.00000E+00	.00000E+00	.00000E+00	.47570E+06	.00000E+00
.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00

THIS MATRIX APPLIES TO THE FOLLOWING PILES -

1

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* * * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   15
* * * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE   16
* * * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE   17
* * * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE   18
* * * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE   19
* * * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE   20
* * * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE   21
* * * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE   22
* * * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE   23
* * * * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   24
* * * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE   25
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LENGTH LESS THAN 5T2 FOR PILE   26
* * * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE   27
* * * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE   28
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* * * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   29
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LENGTH LESS THAN 5T2 FOR PILE    30
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LENGTH LESS THAN 5T2 FOR PILE    31
* * * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    32
* * * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    33
* * * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    34
* * * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    35
* * * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    36
* * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE    37
* * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE    38
* * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE    39
* * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE    40
* * * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    41
* * * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    42
* * * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    43
* * * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    44
* * * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    45
* * * * * * * * * * * * * * * * * * * *

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LENGTH LESS THAN 5T2 FOR PILE    46
* * * * * * * * * * * * * * * * * * * *
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* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    47
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LENGTH LESS THAN 5T2 FOR PILE    48
* * * * * * * * * * * * * * * * * * * *


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LENGTH LESS THAN 5T2 FOR PILE    49
* * * * * * * * * * * * * * * * * * * *


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LENGTH LESS THAN 5T2 FOR PILE    50
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    51
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    52
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    53
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    54
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    55
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE    56
* * * * * * * * * * * * * * * * * * * *


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PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	-18.00	2.00	.00	V	.00	20.00	F
2	-12.00	2.00	.00	V	.00	20.00	F
3	-6.00	2.00	.00	V	.00	20.00	F
4	.00	2.00	.00	V	.00	20.00	F
5	6.00	2.00	.00	V	.00	20.00	F
6	12.00	2.00	.00	V	.00	20.00	F
7	18.00	2.00	.00	V	.00	20.00	F
8	-18.00	6.00	.00	V	.00	20.00	F
9	-12.00	6.00	.00	V	.00	20.00	F
10	-6.00	6.00	.00	V	.00	20.00	F
11	.00	6.00	.00	V	.00	20.00	F
12	6.00	6.00	.00	V	.00	20.00	F
13	12.00	6.00	.00	V	.00	20.00	F
14	18.00	6.00	.00	V	.00	20.00	F
15	-20.50	11.50	.00	V	.00	15.00	F
16	-19.50	11.50	.00	V	.00	15.00	F
17	-18.50	11.50	.00	V	.00	15.00	F
18	-17.50	11.50	.00	V	.00	15.00	F
19	-16.50	11.50	.00	V	.00	15.00	F
20	-15.50	11.50	.00	V	.00	15.00	F
21	-14.50	11.50	.00	V	.00	15.00	F
22	-13.50	11.50	.00	V	.00	15.00	F
23	-12.50	11.50	.00	V	.00	15.00	F
24	-11.50	11.50	.00	V	.00	15.00	F
25	-10.50	11.50	.00	V	.00	15.00	F
26	-9.50	11.50	.00	V	.00	15.00	F
27	-8.50	11.50	.00	V	.00	15.00	F
28	-7.50	11.50	.00	V	.00	15.00	F
29	-6.50	11.50	.00	V	.00	15.00	F
30	-5.50	11.50	.00	V	.00	15.00	F

31	-4.50	11.50	.00	V	.00	15.00	F
32	-3.50	11.50	.00	V	.00	15.00	F
33	-2.50	11.50	.00	V	.00	15.00	F
34	-1.50	11.50	.00	V	.00	15.00	F
35	-.50	11.50	.00	V	.00	15.00	F
36	.50	11.50	.00	V	.00	15.00	F
37	1.50	11.50	.00	V	.00	15.00	F
38	2.50	11.50	.00	V	.00	15.00	F
39	3.50	11.50	.00	V	.00	15.00	F
40	4.50	11.50	.00	V	.00	15.00	F
41	5.50	11.50	.00	V	.00	15.00	F
42	6.50	11.50	.00	V	.00	15.00	F
43	7.50	11.50	.00	V	.00	15.00	F
44	8.50	11.50	.00	V	.00	15.00	F
45	9.50	11.50	.00	V	.00	15.00	F
46	10.50	11.50	.00	V	.00	15.00	F
47	11.50	11.50	.00	V	.00	15.00	F
48	12.50	11.50	.00	V	.00	15.00	F
49	13.50	11.50	.00	V	.00	15.00	F
50	14.50	11.50	.00	V	.00	15.00	F
51	15.50	11.50	.00	V	.00	15.00	F
52	16.50	11.50	.00	V	.00	15.00	F
53	17.50	11.50	.00	V	.00	15.00	F
54	18.50	11.50	.00	V	.00	15.00	F
55	19.50	11.50	.00	V	.00	15.00	F
56	20.50	11.50	.00	V	.00	15.00	F

						910.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-291.7	295.4	571.7	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.61279E+04	.00000E+00	.00000E+00	.00000E+00	.23837E+06	-.65881E+06
.00000E+00	.55780E+04	.00000E+00	-.20972E+06	.00000E+00	-.50932E-10
.00000E+00	.00000E+00	.29651E+06	.32402E+08	.00000E+00	.00000E+00
.00000E+00	-.20972E+06	.32402E+08	.41301E+10	.00000E+00	-.34925E-09
.23837E+06	.00000E+00	.00000E+00	.00000E+00	.62484E+10	-.24561E+08
-.65881E+06	-.50932E-10	.00000E+00	-.34925E-09	-.24561E+08	.20028E+09

56 PILES 1 LOAD CASES

LOAD CASE 1. NUMBER OF FAILURES = 56. NUMBER OF PILES IN TENSION = 42.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	-.2329E-17	-.5464E-01	.7833E-02	-.6257E-04	.3701E-23	-.2167E-19

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE = 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
2	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*

3	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
4	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
5	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
6	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
7	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
8	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
9	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
10	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
11	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
12	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
13	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
14	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
15	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
16	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
17	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
18	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
19	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
20	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
21	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
22	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
23	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
24	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
25	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
26	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
27	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
28	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
29	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
30	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
31	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
32	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
33	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
34	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
35	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
36	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
37	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
38	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
39	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
40	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
41	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
42	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
43	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
44	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
45	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
46	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
47	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
48	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
49	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
50	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
51	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
52	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
53	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
54	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
55	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
56	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-7.7	42.8	331.6	.0	.0
2	.0	-7.7	42.8	331.6	.0	.0
3	.0	-7.7	42.8	331.6	.0	.0
4	.0	-7.7	42.8	331.6	.0	.0
5	.0	-7.7	42.8	331.6	.0	.0
6	.0	-7.7	42.8	331.6	.0	.0
7	.0	-7.7	42.8	331.6	.0	.0
8	.0	-7.7	22.5	331.6	.0	.0
9	.0	-7.7	22.5	331.6	.0	.0
10	.0	-7.7	22.5	331.6	.0	.0
11	.0	-7.7	22.5	331.6	.0	.0
12	.0	-7.7	22.5	331.6	.0	.0
13	.0	-7.7	22.5	331.6	.0	.0
14	.0	-7.7	22.5	331.6	.0	.0
15	.0	-4.4	-3.9	143.0	.0	.0
16	.0	-4.4	-3.9	143.0	.0	.0
17	.0	-4.4	-3.9	143.0	.0	.0
18	.0	-4.4	-3.9	143.0	.0	.0
19	.0	-4.4	-3.9	143.0	.0	.0
20	.0	-4.4	-3.9	143.0	.0	.0
21	.0	-4.4	-3.9	143.0	.0	.0

22	.0	-4.4	-3.9	143.0	.0	.0
23	.0	-4.4	-3.9	143.0	.0	.0
24	.0	-4.4	-3.9	143.0	.0	.0
25	.0	-4.4	-3.9	143.0	.0	.0
26	.0	-4.4	-3.9	143.0	.0	.0
27	.0	-4.4	-3.9	143.0	.0	.0
28	.0	-4.4	-3.9	143.0	.0	.0
29	.0	-4.4	-3.9	143.0	.0	.0
30	.0	-4.4	-3.9	143.0	.0	.0
31	.0	-4.4	-3.9	143.0	.0	.0
32	.0	-4.4	-3.9	143.0	.0	.0
33	.0	-4.4	-3.9	143.0	.0	.0
34	.0	-4.4	-3.9	143.0	.0	.0
35	.0	-4.4	-3.9	143.0	.0	.0
36	.0	-4.4	-3.9	143.0	.0	.0
37	.0	-4.4	-3.9	143.0	.0	.0
38	.0	-4.4	-3.9	143.0	.0	.0
39	.0	-4.4	-3.9	143.0	.0	.0
40	.0	-4.4	-3.9	143.0	.0	.0
41	.0	-4.4	-3.9	143.0	.0	.0
42	.0	-4.4	-3.9	143.0	.0	.0
43	.0	-4.4	-3.9	143.0	.0	.0
44	.0	-4.4	-3.9	143.0	.0	.0
45	.0	-4.4	-3.9	143.0	.0	.0
46	.0	-4.4	-3.9	143.0	.0	.0
47	.0	-4.4	-3.9	143.0	.0	.0
48	.0	-4.4	-3.9	143.0	.0	.0
49	.0	-4.4	-3.9	143.0	.0	.0
50	.0	-4.4	-3.9	143.0	.0	.0
51	.0	-4.4	-3.9	143.0	.0	.0
52	.0	-4.4	-3.9	143.0	.0	.0
53	.0	-4.4	-3.9	143.0	.0	.0
54	.0	-4.4	-3.9	143.0	.0	.0
55	.0	-4.4	-3.9	143.0	.0	.0
56	.0	-4.4	-3.9	143.0	.0	.0

NO FILES WERE GENERATED DURING THIS RUN.
Stop - Program terminated.

Pile Loading (CPGA OUTPUT)

$\text{PileLoad1} := \frac{-3.9\text{kip}}{\text{ft}} \cdot S$ $\quad \text{PileLoad1} = -23.4 \cdot \text{kip}$ $\quad \text{PileM1} := 143 \frac{\text{kip} \cdot \text{in}}{\text{ft}} \cdot S$ $\quad \text{PileM1} = 858 \cdot \text{kip} \cdot \text{in}$

PileLoad 1 and Msht, These numbers are the output from CPGA for sheet piling (continuous) multiplied by the bearing pile spacing to get loading comparable to sheet piles for pile cap calcs

$\text{PileLoad2} := 22.5\text{kip}$ $\quad \text{PileM2} := 331.6\text{kip} \cdot \text{in}$

$\text{PileLoad3} := 42.8\text{kip}$ $\quad \text{PileM3} := 331.6\text{kip} \cdot \text{in}$

$\text{PileLoad4} := 0\text{kip}$ $\quad \text{PileM4} := 0\text{kip} \cdot \text{in}$

$\text{PileLoad5} := 0\text{kip}$ $\quad \text{PileM5} := 0\text{kip} \cdot \text{in}$

Check Vertical Loading

$$\Sigma\text{Vertical} - \frac{\text{PileLoad1} + \text{PileLoad2} + \text{PileLoad3} + \text{PileLoad4} + \text{PileLoad5}}{S} = 0.05 \cdot \frac{\text{kip}}{\text{ft}}$$

Checks if vertical loading calculated above and input into CPGA matches the vertical load output from CPGA. Checks if the equation above is approx. = 0.

EVALUATION OF ALLOWABLE LOAD RESULTS:

CONCRETE SHEET PILE:

Concrete Sheet Pile STRENGTH Mathcad Output:

$FS_{sheetpile} := 1.9$

$Comment_P :=$ "Pile Capacity"

CONCRETE PILING:

Concrete Pile STRENGTH Mathcad Output:

$FS_{pile} := 3.6$

$Comment_P :=$ "Pile Capacity"

See Pile Capacity Calculation for full pile check:

Check Pile loads against geotechnical allowable pile capacities:

For 18" piles:

$AC_{allow} := 37.17kip$ $\quad$ $PileLoad2 = 22.5 \cdot kip$ $\quad$ $PileLoad3 = 42.8 \cdot kip$

$AT_{allow} := 20.25kip$

$$FS_{PileG} := \frac{AC_{allow}}{\max(PileLoad2, PileLoad3)} = 0.87$$

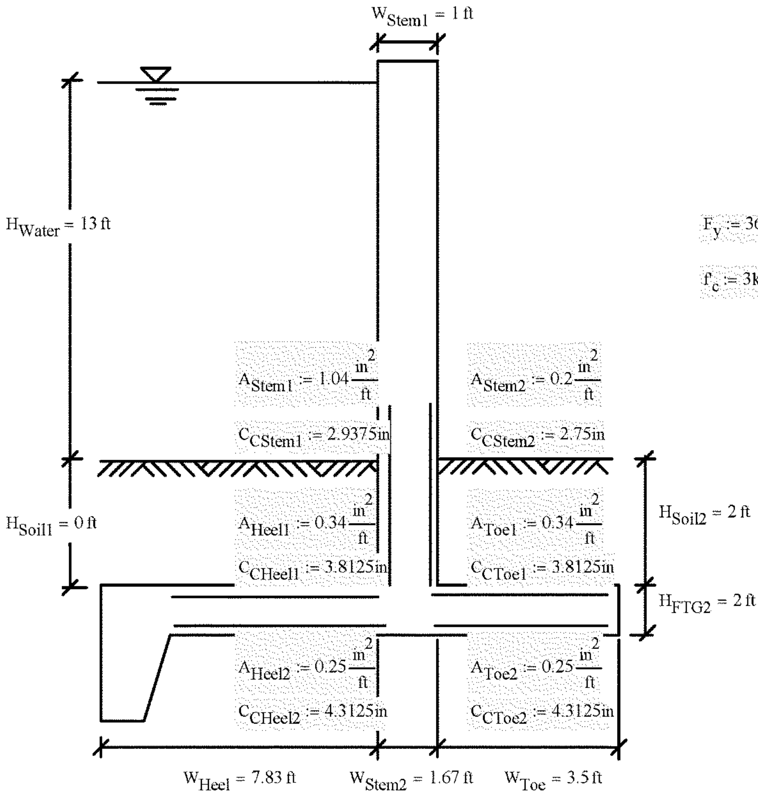
For sheet piles:

$AC_{allow} := 13.43 \frac{kip}{ft}$

$AT_{allow} := 10.10 \frac{kip}{ft}$ $\quad$ $Pile_{sheet} := \frac{PileLoad1}{S} = -3.9 \cdot \frac{kip}{ft}$

$$FS_{SheetPileG} := \frac{\left|AT_{allow}\right|}{\left|\min\left(Pile_{sheet}\right)\right|} = 2.59$$

Typical Properties



$F_y := 36ksi$

$f'_c := 3ksi$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength	$\phi_V := 1$
Flexural Strength	$\phi_B := 1$

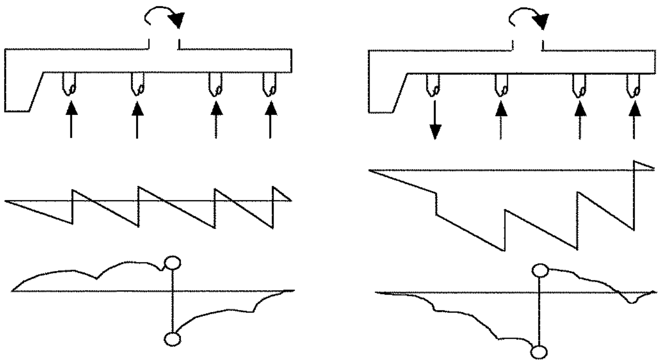
Load Factors

Dead and Live Load Factor	$\gamma_L := 1$
Hydraulic Load Factor	$\gamma_H := 1.0$
Extreme Case Factor	$\gamma_X := 1.0$

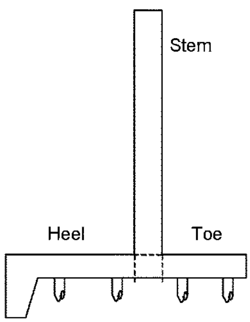
Note: Strength reduction and load factors not used in analysis of existing conditions.

Assumptions for Analysis

It has been assumed that in general the maximum bending moment from loading will occur at or near the connection of the pile cap and stem. As a result section cuts at the intersection of the pile cap and stem wall have been made to evaluate the floodwalls flexural capacity.



Typical Shear and Moment Diagrams



Section Cuts

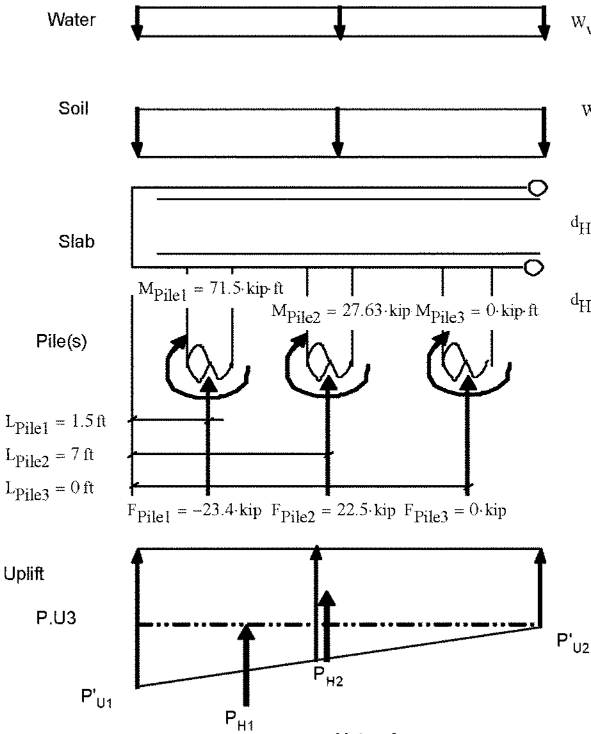
- HEEL Strength Check:

$F_{Pile1} := \text{if}\left(L_{FTG} - \text{Pile1} < W_{Heel}, \text{PileLoad1}, 0\text{kip}\right)$ $L_{Pile1} := \text{if}\left(L_{FTG} - \text{Pile1} < W_{Heel}, L_{FTG} - \text{Pile1}, 0\text{ft}\right)$

$F_{Pile2} := \text{if}\left(L_{FTG} - \text{Pile2} < W_{Heel}, \text{PileLoad2}, 0\text{kip}\right)$ $L_{Pile2} := \text{if}\left(L_{FTG} - \text{Pile2} < W_{Heel}, L_{FTG} - \text{Pile2}, 0\text{ft}\right)$

$F_{Pile3} := \text{if}\left(L_{FTG} - \text{Pile3} < W_{Heel}, \text{PileLoad3}, 0\text{kip}\right)$ $L_{Pile3} := \text{if}\left(L_{FTG} - \text{Pile3} < W_{Heel}, L_{FTG} - \text{Pile3}, 0\text{ft}\right)$

$M_{Pile1} := \text{if}\left(F_{Pile1} = 0, 0, \text{PileM1}\right)$ $M_{Pile2} := \text{if}\left(F_{Pile2} = 0, 0, \text{PileM2}\right)$ $M_{Pile3} := \text{if}\left(F_{Pile3} = 0, 0, \text{PileM3}\right)$



$W_w := \gamma_w \cdot H_{Water} \cdot W_{Heel}$

$W_w = 6354 \frac{\text{lb}}{\text{ft}}$

$W_{sH} := \gamma \cdot H_{Soil1} \cdot W_{Heel}$

$W_{sH} = 0 \frac{\text{lb}}{\text{ft}}$

$d_{H1} := H_{FTG2} - C_{CHeel1}$

$d_{H2} := H_{FTG2} - C_{CHeel2}$

Pile Spacing (O.C.)

$S = 6\,\text{ft}$

$W_{Heel} = 7.83\,\text{ft}$

Note: Assumes Sheet Pile reduces uplift press. by half.

$p_{U3} = -1497.6 \frac{\text{lb}}{\text{ft}}$

$P'_{U1} := p_{U2} = -624 \frac{\text{lb}}{\text{ft}^2}$ $P'_{U2} := \frac{L_{FTG} - W_{Heel}}{\left(L_{FTG} - \frac{W_{CO}}{2}\right)} \cdot \left(p_{U2} - p_{U1}\right) + p_{U1} = -471.01 \frac{\text{lb}}{\text{ft}^2}$

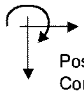
$P_{H1} := \frac{1}{2}\left(P'_{U1} - P'_{U2}\right) \cdot \left(W_{Heel} - \frac{W_{CO}}{2}\right) = -484.46 \frac{\text{lb}}{\text{ft}}$ $P_{H2} := P'_{U2} \cdot \left(W_{Heel} - \frac{W_{CO}}{2}\right) = -2983.07 \frac{\text{lb}}{\text{ft}}$

- HEEL Strength Check (cont'd):

Loading:

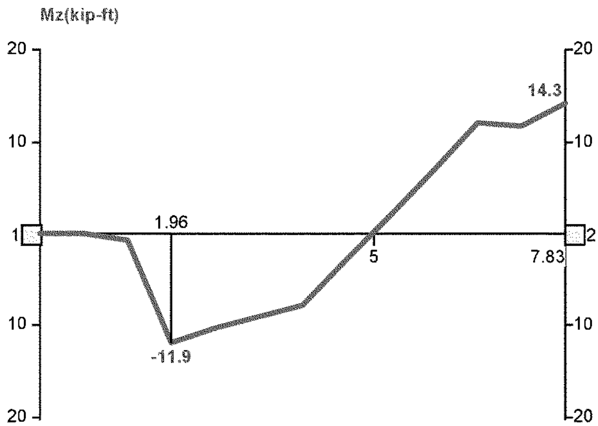
Slab Weight $W_H := \left(W_{Heel} \cdot H_{FTG2}\right) \cdot \gamma_c = 2349.99 \frac{lb}{ft}$

Bending



Positive Sign Convention

ALSO NEED TO CHECK WHERE PILE IS CAUSING LARGE MOMENTS IN THE PILE CAP AT THE PILE

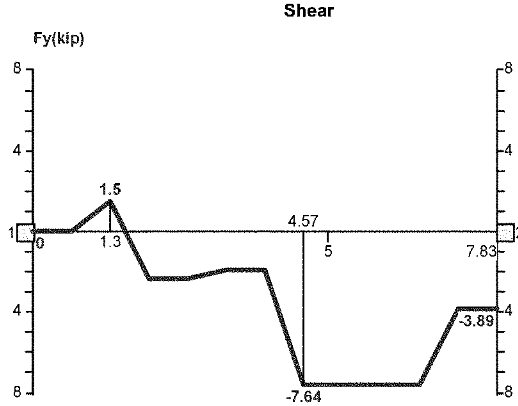


$M_{Htop} := 14.3 \frac{kip \cdot ft}{ft}$

MOMENT DIAGRAM FROM STAAD

$M_{Hbottom} := 11.9 \frac{kip \cdot ft}{ft}$ Loads were input into Staad for whole moment diagram. This moment will cause tension in the steel in the BOTTOM of the heel.

- HEEL Strength Check (cont'd):



SHEAR DIAGRAM FROM STAAD

$V_H := 7.64 \cdot \frac{kip}{ft}$

$V_u := |V_H|$

$V_u = 7.64 \cdot \frac{kip}{ft}$

Capacity:

Flexural Capacity

$A_{s1} := A_{Heel1}$ $\qquad$ $A_{s2} := A_{Heel2}$

$b := 12 \frac{in}{ft}$ $\qquad$ $b := 12 \frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 0.4 \cdot in$ $\qquad$ $a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a2 = 0.29 \cdot in$

$\phi M_{H1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{H1} - \frac{a1}{2}\right)$ $\qquad$ $\phi M_{H2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{H2} - \frac{a2}{2}\right)$

$\phi M_{H1} = 20.39 \cdot \frac{kip \cdot ft}{ft}$ $\qquad$ $\phi M_{H2} = 14.66 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f'_c \cdot psi}$ $\qquad$ $\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{H2} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c1} = 26537 \frac{lb}{ft}$ $\qquad$ $\phi V_{c2} = 25880 \frac{lb}{ft}$

$\phi V_H := \min\left[\phi V_{c1}, \left(\phi V_{c2}\right)\right]$ $\qquad$ $\phi V_H = 25.88 \cdot \frac{kip}{ft}$

- HEEL Strength Check (cont'd):

Factors of Safety

Bending-top of heel

$$\phi M_{H1} = 20.39 \cdot \frac{kip \cdot ft}{ft} \qquad FS_{m1} := \frac{\phi M_{H1}}{|M_{Htop}|}$$

$$M_{Htop} = 14.3 \cdot \frac{kip \cdot ft}{ft}$$

$FS_{m1} = 1.43$

$$Check1 := if(\phi M_{H1} > 1.5M_{Htop}, \text{"OKAY"}, \text{"NO GOOD"})$$

$Check1 = \text{"NO GOOD"}$

Bending-bottm of heel

$$\phi M_{H2} = 14.66 \cdot \frac{kip \cdot ft}{ft} \qquad FS_{m2} := \frac{\phi M_{H2}}{M_{Hbottom}}$$

$FS_{m2} = 1.23$

$$M_{Hbottom} = 11.9 \frac{1}{ft} \cdot kip \cdot ft$$

$$Checkm := if(\phi M_{H2} > 1.5M_{Hbottom}, \text{"OKAY"}, \text{"NO GOOD"})$$

$Checkm = \text{"NO GOOD"}$

$$FS_1 := min(FS_{m1}, FS_{m2})$$

$FS_1 = 1.23$

Shear

$$\phi V_H = 25.88 \cdot \frac{kip}{ft}$$

$$FS_2 := \frac{\phi V_H}{V_u}$$

$$V_u = 7.64 \cdot \frac{kip}{ft}$$

$FS_2 = 3.39$

$$Check2 := if(\phi V_H > 1.5V_u, \text{"OKAY"}, \text{"NO GOOD"})$$

$Check2 = \text{"OKAY"}$

Controlling Factor of Safety

$$FS_H := min(FS_1, FS_2)$$

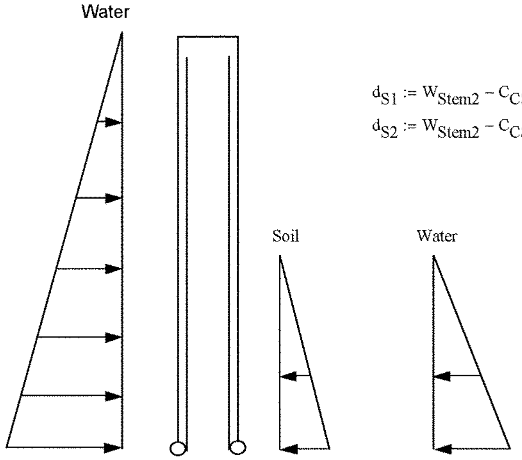
$FS_H = 1.23$

Controlling Mechanism

$$Comment_H := if(FS_1 > FS_2, \text{"Shear in Heel"}, if(FS_{m1} > FS_{m2}, \text{"Flexural Bottom Steel in Heel"}, \text{"Flexural Top Steel in Heel"}))$$

$Comment_H = \text{"Flexural Bottom Steel in Heel"}$

- STEM Strength Check



$d_{S1} := W_{Stem2} - C_{CStem1}$

$d_{S2} := W_{Stem2} - C_{CStem2}$

$P_{ws1} := (H_{Water} + H_{Soil1}) \cdot \gamma_w$

$P_{ws1} = 811 \dfrac{lb}{ft^2}$

$P_{ss1} := K_u \cdot (H_{Soil1}) \cdot (\gamma - \gamma_w)$

$P_{ss1} = 0 \dfrac{lb}{ft^2}$

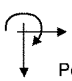
$P_{ss} := H_{Soil2} \cdot K_u \cdot (\gamma - \gamma_w)$ $\quad P_{ws2} := H_{Soil2} \cdot \gamma_w$

$P_{ss} = 65.79 \dfrac{lb}{ft^2}$ $\quad P_{ws2} = 124.8 \dfrac{lb}{ft^2}$

Loading:

Bending

$$M_S := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{P_{ws1} \cdot (H_{Water} + H_{Soil1})}{2} \cdot \frac{H_{Water} + H_{Soil1}}{3} + \frac{P_{ss1} \cdot (H_{Soil1})}{2} \cdot \frac{H_{Soil1}}{3} - \frac{(P_{ss} + P_{ws2}) \cdot (H_{Soil2})}{2} \cdot \frac{H_{Soil2}}{3} \right]$$



Positive Sign Convention

$M_S = 23 \cdot \dfrac{kip \cdot ft}{ft}$ $\quad M_{uS} := |M_S|$

$M_{uS} = 23 \cdot \dfrac{kip \cdot ft}{ft}$

Shear

$$V_S := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{P_{ws1} \cdot (H_{Water} + H_{Soil1})}{2} - \frac{(P_{ss} + P_{ws2}) \cdot (H_{Soil2})}{2} \right]$$

$V_S = 5.08 \cdot \dfrac{kip}{ft}$ $\quad V_{uS} := |V_S|$

$V_{uS} = 5.08 \cdot \dfrac{kip}{ft}$

- STEM Strength Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$

$b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 1.22 \cdot in$

$\phi M_{S1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{S1} - \frac{a1}{2}\right)$

$\phi M_{S1} = 51.3 \cdot \frac{kip \cdot ft}{ft}$

$A_{s2} := A_{Stem2}$

$b := 12\frac{in}{ft}$

$a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a2 = 0.24 \cdot in$

$\phi M_{S2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{S2} - \frac{a2}{2}\right)$

$\phi M_{S2} = 10.27 \cdot \frac{kip \cdot ft}{ft}$

$A_{Stem2} = 0.2 \frac{1}{ft} \cdot in^2$

$d_{S2} = 1.44\, ft$

$\phi M_S := if(M_S > 0, \phi M_{S1}, \phi M_{S2})$ $\quad \phi M_S = 51.3 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{S1} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c1} = 22419 \frac{lb}{ft}$

$\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{S2} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c2} = 22665 \frac{lb}{ft}$

$\phi V_S := if(M_S > 0, \phi V_{c1}, \phi V_{c2})$ $\quad \phi V_S = 22.42 \cdot \frac{kip}{ft}$

Factors of Safety

Bending

$\phi M_S = 51.3 \cdot \frac{kip \cdot ft}{ft}$

$M_{uS} = 22.72 \cdot \frac{kip \cdot ft}{ft}$

$FS_3 := \frac{\phi M_S}{M_{uS}}$ $\quad FS_3 = 2.26$

Check3 := if($\phi M_S > 1.5 M_{uS}$, "OKAY" , "NO GOOD") $\quad$ Check3 = "OKAY"

Shear

$\phi V_S = 22.42 \cdot \frac{kip}{ft}$

$V_{uS} = 5.08 \cdot \frac{kip}{ft}$

$FS_4 := \frac{\phi V_S}{V_{uS}}$ $\quad FS_4 = 4.41$

Check4 := if($\phi V_S > 1.5 V_{uS}$, "OKAY" , "NO GOOD") $\quad$ Check4 = "OKAY"

Controlling Factor of Safety

$FS_S := \min(FS_3, FS_4)$ $\quad FS_S = 2.26$

Controlling Mechanism

$Comment_S$:= if($FS_3 > FS_4$, "Shear in Stem" , if($M_S > 0$, "Stem Flexural Riverside Steel" , "Stem Flexural Landside Steel"))

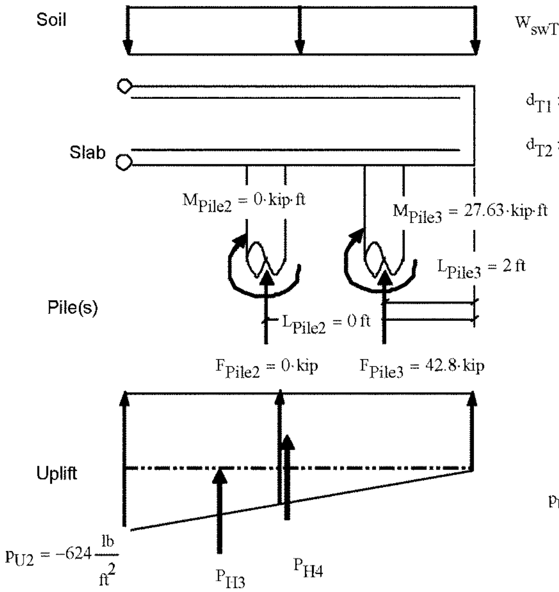
$Comment_S$ = "Stem Flexural Riverside Steel"

- TOE Strength Check

$F_{Pile2} := \text{if}(\text{Pile2} < W_{Toe}, \text{PileLoad2}, 0\text{kip})$ $F_{Pile3} := \text{if}(\text{Pile3} < W_{Toe}, \text{PileLoad3}, 0\text{kip})$ $F_{Pile4} := \text{if}(\text{Pile4} < W_{Toe}, \text{PileLoad4}, 0\text{kip})$

$L_{Pile2} := \text{if}(\text{Pile2} < W_{Toe}, \text{Pile2}, 0\text{ft})$ $L_{Pile3} := \text{if}(\text{Pile3} < W_{Toe}, \text{Pile3}, 0\text{ft})$ $L_{Pile4} := \text{if}(\text{Pile4} < W_{Toe}, \text{Pile4}, 0\text{ft})$

$M_{Pile2} := \text{if}(\text{Pile2} < W_{Toe} + 6\text{in}, \text{PileM2}, 0\text{kip})$ $M_{Pile3} := \text{if}(F_{Pile3} = 0, 0, \text{PileM3})$ $M_{Pile4} := \text{if}(F_{Pile4} = 0, 0, \text{PileM4})$



$W_{swT} := (\gamma \cdot H_{Soil2}) \cdot W_{Toe}$ $\quad W_{swT} = 805 \frac{\text{lb}}{\text{ft}}$

$d_{T1} := H_{FTG2} - C_{CToe1}$ $\quad W_{Toe} = 3.5\,\text{ft}$

$d_{T2} := H_{FTG2} - C_{CToe2}$

Pile Spacing (O.C.)

$M_{Pile4} = 0 \cdot \text{kip} \cdot \text{ft}$ $\quad S = 6\,\text{ft}$

$L_{Pile4} = 0$

Note: Assumes Sheet Piling reduces uplift pressures by half.

$p_{U1} = -346.22 \frac{\text{lb}}{\text{ft}^2}$

$$P'_{U3} := -\left[\frac{W_{Toe}}{L_{FTG} - \frac{W_{CO}}{2}} \cdot \left(|p_{U2}| - |p_{U1}|\right) + |p_{U1}|\right] \qquad P'_{U3} = -430.77 \frac{\text{lb}}{\text{ft}^2}$$

$$P_{H3} := -\left[\frac{1}{2} \cdot \left(|P'_{U3}| - |p_{U1}|\right) \cdot W_{Toe}\right] \qquad P_{H3} = -147.96 \frac{\text{lb}}{\text{ft}}$$

$$P_{H4} := p_{U1} \cdot W_{Toe} \qquad P_{H4} = -1211.78 \frac{\text{lb}}{\text{ft}}$$

Loading:

Slab Weight $\quad W_T := (W_{Toe} \cdot H_{FTG2}) \cdot \gamma_c \qquad W_T = 1050 \frac{\text{lb}}{\text{ft}}$

Pile Centroid

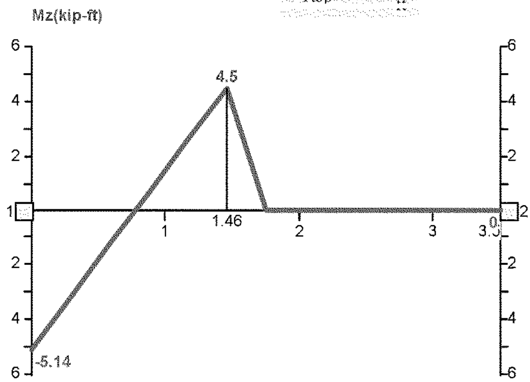
$$L_{PileT} := \frac{L_{Pile2} \cdot F_{Pile2} + L_{Pile3} \cdot F_{Pile3} + L_{Pile4} \cdot F_{Pile4}}{F_{Pile2} + F_{Pile3} + F_{Pile4}} \qquad L_{PileT} = 2\,\text{ft}$$

- TOE Strength Check (Cont'd)

Bending Positive Sign Convention

ALSO NEED TO CHECK WHERE PILE IS CAUSING LARGE MOMENTS IN THE PILE CAP AT THE PILE

$M_{Ttop} := 4.5 \frac{kip \cdot ft}{ft}$

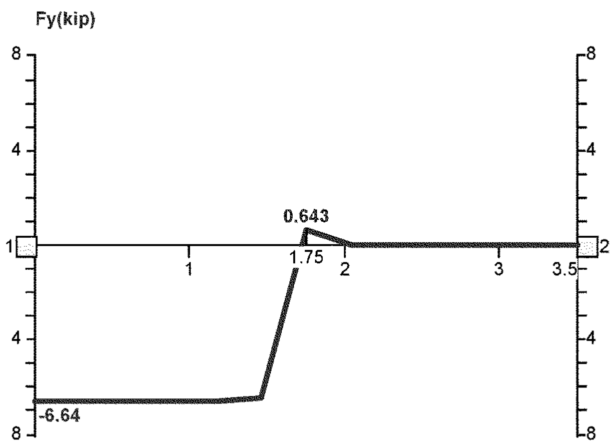


MOMENT DIAGRAM FROM STAAD

$M_{Tbottom} := 5.4 \frac{kip \cdot ft}{ft}$

Loads were input into Staad for whole moment diagram. This moment will cause tension in the steel in the BOTTOM of the heel.

Shear



$V_T := 6.64 \frac{kip}{ft}$

$V_{uT} := |V_T|$

$V_{uT} = 6.64 \frac{kip}{ft}$

- TOE Strength Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Toe1}$

$b := 12 \frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 0.4 \cdot in$

$\phi M_{T1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{T1} - \frac{a1}{2} \right)$

$\phi M_{T1} = 20.39 \cdot \frac{kip \cdot ft}{ft}$

$A_{s2} := A_{Toe2}$

$b := 12 \frac{in}{ft}$

$a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a2 = 0.29 \cdot in$

$\phi M_{T2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{T2} - \frac{a2}{2} \right)$

$\phi M_{T2} = 14.66 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{T1} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c1} = 26537 \frac{lb}{ft}$

$\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{T2} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c2} = 25880 \frac{lb}{ft}$

$\phi V_T := \min\left[\left(\phi V_{c1} \right), \phi V_{c2} \right]$ $\quad \phi V_T = 25.88 \cdot \frac{kip}{ft}$

- TOE Strength Check (con'td)

Factors of Safety

Bending - top of toe

$$\phi M_{T1} = 20.39 \cdot \frac{kip \cdot ft}{ft} \qquad FS_{m1} := \frac{\phi M_{T1}}{M_{Ttop}}$$

$$M_{Ttop} = 4.5 \cdot \frac{kip \cdot ft}{ft} \qquad FS_{m1} = 4.53$$

$$Check5 := if(\phi M_{T1} > 1.5M_{Ttop}, "OKAY", "NO\ GOOD") \qquad Check5 = "OKAY"$$

Bending - bottom of toe

$$\phi M_{T2} = 14.66 \cdot \frac{kip \cdot ft}{ft} \qquad FS_{m2} := \frac{\phi M_{T2}}{M_{Tbottom}} \qquad FS_{m2} = 2.71$$

$$M_{Tbottom} = 5.4 \frac{1}{ft} \cdot kip \cdot ft$$

$$Checkm := if(\phi M_{T2} > 1.5M_{Tbottom}, "OKAY", "NO\ GOOD") \qquad Checkm = "OKAY"$$

$$FS_5 := \min(FS_{m1}, FS_{m2}) \qquad FS_5 = 2.71$$

Shear

$$\phi V_T = 25.88 \cdot \frac{kip}{ft} \qquad FS_6 := \frac{\phi V_T}{V_{uT}}$$

$$V_{uT} = 6.64 \cdot \frac{kip}{ft} \qquad FS_6 = 3.9$$

$$Check6 := if(\phi V_T > 1.5V_{uT}, "OKAY", "NO\ GOOD") \qquad Check6 = "OKAY"$$

Controlling Factor of Safety

$$FS_T := \min(FS_5, FS_6) \qquad FS_T = 2.71$$

Controlling Mechanism

$$Comment_T := if(FS_5 > FS_6, "Shear\ in\ Heel", if(FS_{m1} < FS_{m2}, "Flexural\ Top\ Steel\ in\ Toe", "Flexural\ Bottom\ Steel\ in\ Toe"))$$

$$Comment_T = "Flexural\ Bottom\ Steel\ in\ Toe"$$

Overall Factor of Safety

Strength Factor of Safety

Sheet Pile Factor of Safety, $FS_{sheetpile} = 1.9$

Pile Factor of Safety, $FS_{pile} = 3.6$

Heel Factor of Safety, $FS_H = 1.23$

Stem Factor of Safety, $FS_S = 2.26$

Toe Factor of Safety, $FS_T = 2.71$

The strength FS for the piles should be greater than or equal to 1.5, if not a reliability analysis needs to be completed for the piles.

$$FoS := \min\left(FS_{sheetpile}, FS_{pile}, FS_H, FS_S, FS_T\right)$$

Limiting Mechanism

$Mechanism := Comment_P$

$Mechanism := if\left(FoS = FS_H, Comment_H, Mechanism\right)$

$Mechanism := if\left(FoS = FS_S, Comment_S, Mechanism\right)$

$Mechanism := if\left(FoS = FS_T, Comment_T, Mechanism\right)$

$FoS = 1.23$ Mechanism = "Flexural Bottom Steel in Heel"

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis.

Geotechnical Capacity Factor of Safety:

Piles $FS_{PileG} = 0.87$

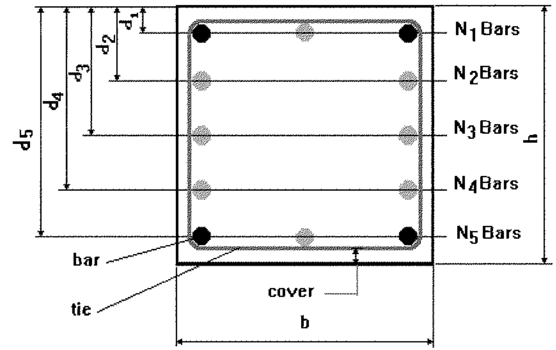
Sheet Piles $FS_{SheetPileG} = 2.59$

Pile load is GREATER than the allowable geotechnical capacity.
->Reliability analysis is REQUIRED for geotechnical based axial capacity.

Units:

$kips := 1000lb$ $psf := \frac{lb}{ft^2}$ $psi := \frac{lb}{in^2}$ $ksi := \frac{kips}{in^2}$ $pcf := \frac{lb}{ft^3}$ $plf := \frac{lb}{ft}$

$ton := 2000lb$ $tsf := \frac{ton}{ft^2}$ $klf := \frac{kips}{ft}$



This program calulates the column interaction curve for a rectangular concrete column using the methods of the Concrete Reinforcing Steel Institute. Note this program also accounts for less than 1 percent steel by reducing the width. Procedure based on CRSI Manual Pg 3-5 to Pg 3-8

$h := 15in$	Column Depth	$cover := 2.25in$	Cover -in	
$w := 15in$	Column Width	$N_1 := 3$	Bars in Row 1	
$bar := 7$	Bar Size	$N_2 := 2$	Bars in Row 2	
$tie := 2$	Tie Size	$N_3 := 3$	Bars in Row 3	
$f'c := 4ksi$	Conc. Str. - ksi	$N_4 := 0$	Bars in Row 4	
$fy := 40ksi$	Bar Yield - ksi	$N_5 := 0$	Bars in Row 5	$row \equiv 3$

Total Rows of Bars- NOTE: This can be changed and more rows added if required.

Axial Compression with Aci Reduction value: $\phi := 0.7$ <=== Original Code Reduction Value

$\phi := 1$ <=Reduction value used F.S. analysis

Modulus of Elasticty: $Es := 29000ksi$

Concrete Strength: $f'c := 4000psi$

FEMA 310 Allowed for concrete aging: $FEMA\_Factor := 1.00$

Nominal Concrete Strength assumed orginally used: $f'c := f'c \cdot FEMA\_Factor$ $f'c = 4000 \cdot psi$

NOTE - if FEMA factor is 1.00 then concrete strength is the nominal concrete strength. If trying to get the expected concrete strength the FEMA factor should be 1.25. See FEMA 310 guidance.

$areabar \equiv (0in^2\ \ 0in^2\ \ .05in^2\ \ 0.11in^2\ \ 0.2in^2\ \ 0.31in^2\ \ 0.44in^2\ \ 0.6in^2\ \ 0.79in^2\ \ 1in^2\ \ 1.27in^2\ \ 1.56in^2\ \ 0in^2\ \ 0in^2\ \ 2.25in^2)$

$diabar \equiv (0in\ \ 0in\ \ .25in\ \ 0.38in\ \ 0.5in\ \ 0.63in\ \ 0.75in\ \ 0.88in\ \ 1in\ \ 1.13in\ \ 1.27in\ \ 1.41in\ \ 0in\ \ 0in\ \ 1.69in)$

Calculate effective stress block per ACI 318 sect. 10.2.7.3 $\beta 1 := \text{if}\left[f'c \le 4ksi, 0.85, \text{if}\left[f'c > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'c}{ksi} - 4\right)\right]\right]$ $\beta 1 = 0.85$

Iterations $i := 1..row$

Bar Diameter $db := diabar_{0,bar}$ $db = 0.88\cdot in$

Bar area at i $A_i := N_i \cdot areabar_{0,bar}$

Tie Diameter $dt := diabar_{0,tie}$ $dt = 0.250\cdot in$

Distances from compression face $d_1 := cover + \frac{db}{2} + dt$ $d_1 = 2.94\cdot in$

Spacing between rows of bars $space := \frac{h - 2\cdot d_1}{row - 1}$ $space = 4.56\cdot in$ $d_i := d_1 + (i-1)\cdot space$

$i =$	$N_i =$	$A_i =$ (in^2)	$d_i =$ (in)
1	3	1.8	2.94
2	2	1.2	7.5
3	3	1.8	12.06

Gross Area: $Ag := w\cdot h$ $Ag = 225\cdot in^2$

Steel Area $Ast := \sum_{i=1}^{row} A_i$ $Ast = 4.8\cdot in^2$

Reduce width if $\rho < 0.01$: $\frac{Ast}{Ag} = 0.0213$ $b := \text{if}\left(\frac{Ast}{Ag} > 0.01, w, \frac{Ast}{0.01\cdot h}\right)$ Effective Width $b = 15\cdot in$

$Ag := b\cdot h$ $Ag = 225\cdot in^2$ $\frac{Ast}{Ag} = 0.0213$

Calculate Axial Load at no eccentricity, e (min) at $\phi Pn(max)$:

$Pn_1 := 0.8 \cdot [0.85 \cdot f'c \cdot (Ag - Ast) + fy \cdot Ast]$ $Pn_1 = 753 \cdot kips$

Equate as the first set of points on the interaction diagram: $Pn_0 := Pn_1$ $Pn_0 = 752.544 \cdot kips$ $Mn_0 := 0 kips \cdot in$

Axial Compression with Aci Reduction value: $\phi := 1$ $\phi \cdot Pn_1 = 753 \cdot kips$

Assume c=h $c := h$ $c = 15 \cdot in$

Given

$$Pn_1 = \sum_{i=1}^{row} \left[\left[(c - d_i) \cdot \frac{0.003}{c} \cdot Es - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) \right] \cdot A_i \right] + 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$ $Pn_1 = 752.544 \cdot kips$

$c := Find(c)$ $c = 13.37 \cdot in$

Calculate Width of Compression block: $a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $a = 11.362 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$

$\varepsilon s_i =$

0.002
0.001
0

$fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$fs_i =$

40
38.185
8.506

$\cdot ksi$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi\right) \cdot A_i$ $\quad Fs_i = \begin{bmatrix} 65.88 \\ 41.742 \\ 15.311 \end{bmatrix} \cdot kips$

$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$ $\quad Ms_i = \begin{bmatrix} 300.413 \\ 0 \\ -69.817 \end{bmatrix} \cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 230.595 \cdot kips \cdot in$

$Fst := \sum_{i=1}^{row} Fs_i$ $\quad Fst = 122.933 \cdot kips$

$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 87.839 \cdot kips \cdot ft$

$Fc := 0.85 \cdot f'c \cdot \beta1 \cdot c \cdot b$ $\quad Fc = 579.454 \cdot kips$

$Mn_1 := Mc + Mst$ $\quad Mn_1 = 1284.666 \cdot kips \cdot in$

$e_1 := \frac{Mn_1}{Pn_1}$ $\quad e_1 = 1.71 \cdot in$

$d_{row} = 12.06 \cdot in$

Find Pn and Mn at point of zero tension in bars:

$c := d_{row}$ $\quad c = 12.06 \cdot in$ $\quad a := \beta 1 \cdot c$ $\quad a = 10.251 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\qquad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\qquad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$i =$	$\varepsilon s_i =$	$fs_i =$ (ksi)	$Fs_i =$ (kips)	$Ms_i =$ (kips·in)	$A_i =$ (in^2)	$d_i =$ (in)
1	0.002	40	65.88	300.413	1.8	2.94
2	0.001	38.185	41.742	0	1.2	7.5
3	0	8.506	15.311	-69.817	1.8	12.06

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 230.595 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 1241.391 \cdot kips \cdot in$

$Mn_2 := Mc + Mst$ $\quad Mn_2 = 1471.986 \cdot kips \cdot in$ $\qquad Mn_2 = 1472 \cdot kips \cdot in$

$Mn_1 = 1284.666 \cdot kips \cdot in$ $\quad Mn_1 := if(Mn_1 < Mn_2, Mn_1, Mn_2)$ $\quad Mn_1 = 1284.666 \cdot kips \cdot in$

$Pn_2 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_2 = 646 \cdot kips$ $\quad Pn_1 = 752.544 \cdot kips$ $\quad Pn_2 := if(Pn_2 < Pn_1, Pn_2, Pn_1)$ $\qquad Pn_2 = 645.734 \cdot kips$

Find Pn and Mn at point of 0.25 *fy tension in bars:

$$c := \frac{d_{row}}{1 + \frac{0.25 \cdot fy}{Es \cdot 0.003}} \qquad d_{row} = 12.06 \cdot in \qquad a := \beta 1 \cdot c$$

Find Moment:

$$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(Es \cdot \varepsilon s_i > fy, fy, \text{if}\left(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i\right)\right)$$

$$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$$

d_i = (ft)	εs_i =	fs_i = (ksi)	A_i = (in^2)	Fs_i = (kips)	Ms_i = (kips·in)
0.245	0.002	40	1.8	65.88	300.413
0.625	0.001	26.677	1.2	27.932	0
1.005	-0	-10	1.8	-18	82.08

$$Mst := \sum_{i=1}^{row} Ms_i \qquad Mst = 382 \cdot kips \cdot in \qquad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right) \qquad Mc = 1361.182 \cdot kips \cdot in \qquad Mn_3 := Mc + Mst \qquad Mn_3 = 1744 \cdot kips \cdot in$$

$$Pn_3 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b \qquad Pn_3 = 545 \cdot kips$$

Find Pn and Mn at point of 0.50 *fy tension in bars:

$d_{row} = 12.06 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{0.50 \cdot fy}{Es \cdot 0.003}}$ $\quad c = 9.806 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$ (ft)	$\varepsilon s_i =$	$fs_i =$ (ksi)	$A_i =$ (in^2)	$Fs_i =$ (kips)	$Ms_i =$ (kips·in)
0.245	0.002	40	1.8	65.88	300.413
0.625	0.001	20.458	1.2	20.469	0
1.005	-0.001	-20	1.8	-36	164.16

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 465 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 1416.599 \cdot kips \cdot in$ $\quad Mn_4 := Mc + Mst$ $\quad Mn_4 = 1881 \cdot kips \cdot in$

$Pn_4 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_4 = 475 \cdot kips$

Find Pn and Mn at Balanced:

$d_{row} = 12.06 \cdot in$ $c := \dfrac{d_{row}}{1 + \dfrac{fy}{Es \cdot 0.003}}$ $c = 8.262 \cdot in$ $a := \beta1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$ (ft)	$\varepsilon s_i =$	$fs_i =$ (ksi)	$A_i =$ (in^2)	$Fs_i =$ (kips)	$Ms_i =$ (kips·in)
0.245	0.002	40	1.8	65.88	300.413
0.625	0	8.02	1.2	9.624	0
1.005	-0.001	-40	1.8	-72	328.32

$Mst := \sum_{i=1}^{row} Ms_i$ $Mst = 629 \cdot kips \cdot in$ $Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $Mc = 1428.557 \cdot kips \cdot in$ $Mn_5 := Mc + Mst$ $Mn_5 = \blacksquare \cdot kips \cdot in$

$Pn_5 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $Pn_5 = 362 \cdot kips$

Find Mn at ϕ Pn = 0.1*f'c*Ag:

$Pn_6 := \frac{0.1 \cdot f'c \cdot w \cdot h}{\phi}$ $Pn_6 = 90 \cdot kips$ $Pn_6 := if(Pn_6 > Pn_5, Pn_5, Pn_6)$ $Pn_6 = 90 \cdot kips$

Assume c=h/2 $c := h \cdot 0.5$ $c = \blacksquare \cdot in$

Given

$$Pn_6 = \sum_{i=1}^{row} \left[\left[\begin{array}{ll} (c - d_i) \cdot \frac{0.003}{c} \cdot Es \ldots \\ + -if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es < fy \\ fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{array} \right] \cdot A_i \right] \ldots$$
$$+ 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $c = 4.02 \cdot in$

$a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $a = 3.41 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$ (in)	$\varepsilon s_i =$	$fs_i =$ (psi)	$A_i =$ (in^2)	$Fs_i =$ (kips)	$Ms_i =$ (in·kips)
2.94	0.001	23324.942	1.8	35.865	163.544
7.5	-0.003	-40000	1.2	-48	0
12.06	-0.006	-40000	1.8	-72	328.32

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad$ $Mst = 492 \cdot kips \cdot in$

$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad$ $Mc = 1008.729 \cdot kips \cdot in$

$Mn_6 := Mc + Mst$ $\quad$ $Mn_6 = 1501 \cdot kips \cdot in$

$Pn_6 = 90 \cdot kips$

Find Mn at ϕ Pn = 0.0:

$Pn_7 := 0lb$ $\qquad$ $Pn_7 = 0\,lb$

Assume c=0.2h $\qquad$ $c := 0.2\cdot h$

Given

$$Pn_7 = \sum_{i=1}^{row}\left[\left[\begin{array}{ll}(c-d_i)\cdot\frac{0.003}{c}\cdot Es\ \ldots & \text{if } (c-d_i)\cdot\frac{0.003}{c}\cdot Es < fy \\ +\,-\text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'c, 0\right) & \\ fy - \text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'c, 0\right) & \text{if } (c-d_i)\cdot\frac{0.003}{c}\cdot Es \ge fy \\ -fy - \text{if}\left(d_i < \beta1\cdot c, 0.85\cdot f'c, 0\right) & \text{if } (c-d_i)\cdot\frac{0.003}{c}\cdot Es \le -fy\end{array}\right]\cdot A_i\right]\ \ldots$$
$$+\,0.85\cdot f'c\cdot\beta1\cdot c\cdot b$$

$c := \text{Find}(c)$ $\qquad$ $c = 2.864\cdot in$ $\qquad$ $a := \text{if}(\beta1\cdot c > h, h, \beta1\cdot c)$ $\qquad$ $a = 2.434\cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\frac{0.003}{c}$

$fs_i := \text{if}(Es\cdot\varepsilon s_i > fy, fy, \text{if}(Es\cdot\varepsilon s_i < 0 - fy, 0 - fy, Es\cdot\varepsilon s_i))$

$Fs_i := fs_i\cdot A_i - \text{if}(\beta 1\cdot c > d_i, 0.85\cdot f'c, 0ksi)\cdot A_i$

$Ms_i := Fs_i\cdot\left(\frac{h}{2} - d_i\right)$

$d_i =$ (ft)	$\varepsilon s_i =$	$fs_i =$ (ksi)	$A_i =$ (in^2)	$Fs_i =$ (kips)	$Ms_i =$ (kips·in)
0.245	-0	-2.308	1.8	-4.155	-18.946
0.625	-0.005	-40	1.2	-48	0
1.005	-0.01	-40	1.8	-72	328.32

$Mst := \sum_{i=1}^{row} Ms_i$ $\qquad Mst = 309\cdot kips\cdot in$

$Mc := 0.85\cdot f'c\cdot a\cdot w\cdot\left(\frac{h}{2} - \frac{a}{2}\right)$ $\qquad Mc = 780\cdot kips\cdot in$

$Mn_7 := Mc + Mst$ $\qquad Mn_7 = 1089\cdot kips\cdot in$

$j := 0..7 \quad \phi_j := 1$

Results

$\phi_j \cdot Pn_j =$ (·kip)	$\phi_j \cdot Mn_j =$ (·kips·in)
752.544	0
752.544	1284.666
645.734	1471.986
544.716	1743.675
475.43	1881.172
361.643	2057.29
90	1500.593
0	1089.413

Criteria:

Depth $h = 15 \cdot in$ $bar = 7$

Width $w = 15 \cdot in$ $tie = 2$

Eff. Width $b = 15 \cdot in$ $fy = 40 \cdot ksi$

$cover = 2.25 \cdot in$ $f'c = 4000 \cdot psi$

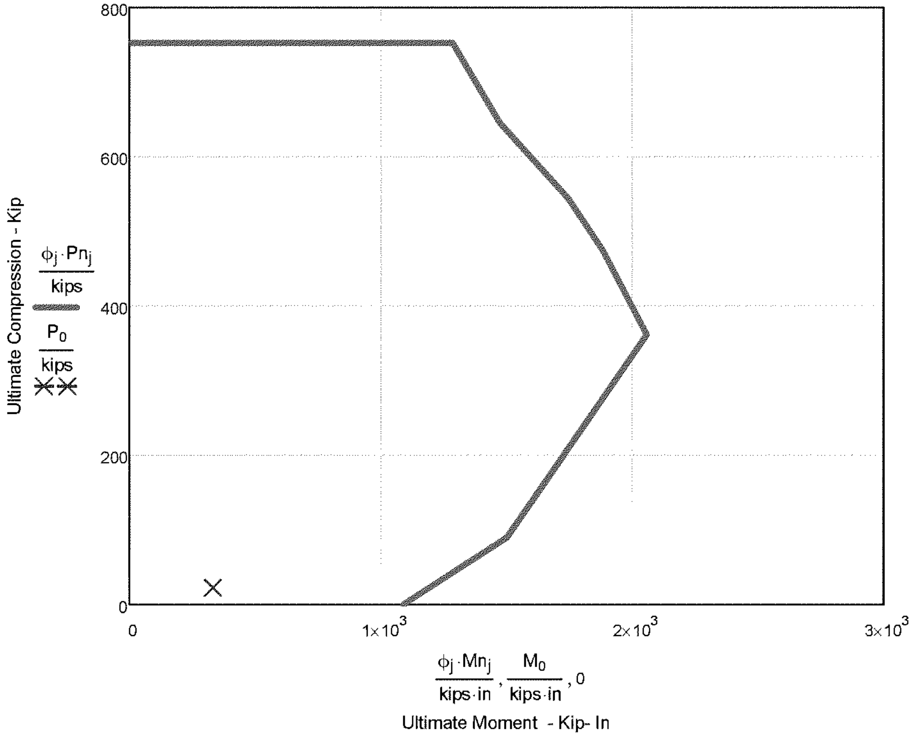
$Ag = 225 \cdot in^2$ $Ast = 4.8 \cdot in^2$

$\frac{Ast}{w \cdot h} = 0.0213$ Actual ratio ρ

$\frac{Ast}{Ag} = 0.0213$ Eff. ratio ρ

Demand Loads Water to Top of Wall:

Axial Load from CPGA output: $P_0 := 22.5\text{kips}$ Moment from CPGA output: $M_0 := 331.6\text{kips}\cdot\text{in}$



Interaction Diagram

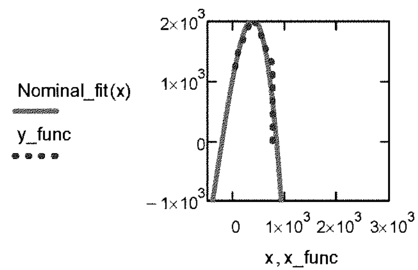
Nominal Curve Used to Find Factor of Safety:

Data Points set to function: $y\_func := \frac{Mn}{kips \cdot in}$ $x\_func := \frac{Pn}{kips}$

Degree of Polynominal Fit: $k := 3$

Returns a vector which interp uses to find the kth order polynomial that best fits the x and y data values

$Nominal := regress(x\_func, y\_func, k)$ $Nominal\_fit(x) := interp(Nominal, x\_func, y\_func, x)$



Strength Factor of Safety:

Read the Corresponding Moment for the demand axial load ($P_0 = 22.5 \cdot kips$)

$M_{capacity} := Nominal\_fit\left(\frac{P_0}{kips}\right)$ $M_{capacity} = 1205.635$

Demand Moment: $M_{demand} := \frac{M_0}{kips \cdot in}$ $M_{demand} = 331.6$

Factor of Safety: $FS_{strength} := \frac{M_{capacity}}{M_{demand}}$ $FS_{strength} = 3.6$

FEMA 310 GUIDANCE

4.2.4.4 Component Strength

Component strength for all actions shall be taken as the expected strength, Q_{CE}. Unless calculated otherwise, the expected strength shall be assumed equal to the nominal strength multiplied by 1.25. Alternatively, if allowable stresses are used, nominal strengths shall be taken as the allowable values multiplied by the following values:

Steel	1.7
Masonry	2.5
Wood	2.0

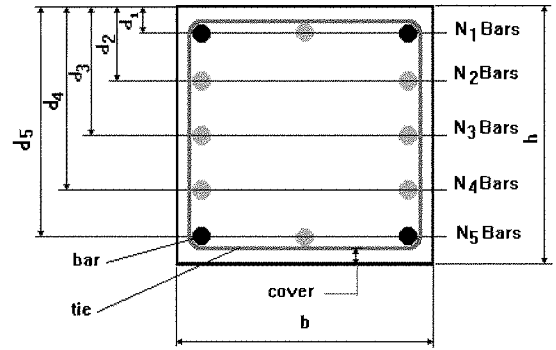
Except for wood diaphragms and wood and masonry shear walls, the allowable values shall not include a one-third increase for short term loading.

When calculating capacities of deteriorated elements, the evaluating design professional shall make

Units:

$kips := 1000lb \quad psf := \frac{lb}{ft^2} \quad psi := \frac{lb}{in^2} \quad ksi := \frac{kips}{in^2} \quad pcf := \frac{lb}{ft^3} \quad plf := \frac{lb}{ft}$

$ton := 2000lb \quad tsf := \frac{ton}{ft^2} \quad klf := \frac{kips}{ft}$



This program calulates the column interaction curve for a rectangular concrete column using the methods of the Concrete Reinforcing Steel Institute. Note this program also accounts for less than 1 percent steel by reducing the width. Procedure based on CRSI Manual Pg 3-5 to Pg 3-8

$h := 10in$	Column Depth	$cover := 2.25in$	Cover -in	
$w := 12in$	Column Width	$N_1 := 2$	Bars in Row 1	
$bar := 6$	Bar Size	$N_2 := 2$	Bars in Row 2	
$tie := 2$	Tie Size	$N_3 := 0$	Bars in Row 3	
$f'c := 4ksi$	Conc. Str. - ksi	$N_4 := 0$	Bars in Row 4	
$fy := 40ksi$	Bar Yield - ksi	$N_5 := 0$	Bars in Row 5	$row \equiv 2$

Total Rows of Bars- NOTE: This can be changed and more rows added if required.

Axial Compression with Aci Reduction value: $\phi := 0.7$ <=== Original Code Reduction Value

$\phi := 1$ <=Reduction value used F.S. analysis

Modulus of Elasticty: $Es := 29000ksi$

Concrete Strength: $f'c := 4000psi$

FEMA 310 Allowed for concrete aging: $FEMA\_Factor := 1.00$

Nominal Concrete Strength assumed orginally used: $f'c := f'c \cdot FEMA\_Factor$ $\boxed{f'c = 4000 \cdot psi}$

NOTE - if FEMA factor is 1.00 then concrete strength is the nominal concrete strength. If trying to get the expected concrete strength the FEMA factor should be 1.25. See FEMA 310 guidance.

$areabar \equiv \begin{pmatrix} 0in^2 & 0in^2 & .05in^2 & 0.11in^2 & 0.2in^2 & 0.31in^2 & 0.44in^2 & 0.6in^2 & 0.79in^2 & 1in^2 & 1.27in^2 & 1.56in^2 & 0in^2 & 0in^2 & 2.25in^2 \end{pmatrix}$

$diabar \equiv (0in \;\; 0in \;\; .25in \;\; 0.38in \;\; 0.5in \;\; 0.63in \;\; 0.75in \;\; 0.88in \;\; 1in \;\; 1.13in \;\; 1.27in \;\; 1.41in \;\; 0in \;\; 0in \;\; 1.69in)$

Calculate effective stress block per ACI 318 sect. 10.2.7.3 $\beta 1 := if\left[f'c \le 4ksi, 0.85, if\left[f'c > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'c}{ksi} - 4\right)\right]\right]$ $\beta 1 = 0.85$

Iterations $i := 1 .. row$ Bar Diameter $db := diabar_{0, bar}$ $db = 0.75\cdot in$

Bar area at i $A_i := N_i \cdot areabar_{0, bar}$ Tie Diameter $dt := diabar_{0, tie}$ $dt = 0.250\cdot in$

Distances from compression face $d_1 := cover + \frac{db}{2} + dt$ $d_1 = 2.875\cdot in$

Spacing between rows of bars $space := \frac{h - 2\cdot d_1}{row - 1}$ $space = 4.25\cdot in$ $d_i := d_1 + (i - 1)\cdot space$

$i = \begin{array}{|c|}\hline 1 \\ \hline 2 \\ \hline\end{array}$ $N_i = \begin{array}{|c|}\hline 2 \\ \hline 2 \\ \hline\end{array}$ $A_i = \begin{array}{|c|}\hline 0.88 \\ \hline 0.88 \\ \hline\end{array} \cdot in^2$ $d_i = \begin{array}{|c|}\hline 2.875 \\ \hline 7.125 \\ \hline\end{array} \cdot in$

Gross Area: $Ag := w\cdot h$ $Ag = 120\cdot in^2$

Steel Area $Ast := \sum_{i=1}^{row} A_i$ $Ast = 1.76\cdot in^2$

Reduce width if $\rho < 0.01$: $\frac{Ast}{Ag} = 0.0147$ $b := if\left(\frac{Ast}{Ag} > 0.01, w, \frac{Ast}{0.01\cdot h}\right)$ Effective Width $b = 12\cdot in$

$Ag := b\cdot h$ $Ag = 120\cdot in^2$ $\frac{Ast}{Ag} = 0.0147$

BY: KSM 12/16/2011

Calculate Axial Load at no eccentricity, e (min) at ϕPn(max):

$Pn_1 := 0.8 \cdot [0.85 \cdot f'c \cdot (Ag - Ast) + fy \cdot Ast]$ $Pn_1 = 378 \cdot kips$

Equate as the first set of points on the interaction diagram: $Pn_0 := Pn_1$ $Pn_0 = 377.933 \cdot kips$ $Mn_0 := 0 kips \cdot in$

Axial Compression with Aci Reduction value: $\phi := 1$ $\phi \cdot Pn_1 = 378 \cdot kips$

Assume c=h $c := h$ $c = 10 \cdot in$

Given

$$Pn_1 = \sum_{i=1}^{row} \left[\left[(c - d_i) \cdot \frac{0.003}{c} \cdot Es - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) \right] \cdot A_i \right] + 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$ $Pn_1 = 377.933 \cdot kips$

$c := Find(c)$ $c = 9.08 \cdot in$

Calculate Width of Compression block: $a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $a = 7.722 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\varepsilon s_i = \begin{bmatrix} 0.002 \\ 0.001 \end{bmatrix}$

$fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$ $fs_i = \begin{bmatrix} 40 \\ 18.769 \end{bmatrix} \cdot ksi$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi\right) \cdot A_i$ $\quad Fs_i = \begin{bmatrix} 32.208 \\ 13.525 \end{bmatrix} \cdot kips$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$ $\quad Ms_i = \begin{bmatrix} 68.442 \\ -28.741 \end{bmatrix} \cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 39.701 \cdot kips \cdot in$ $\quad Fst := \sum_{i=1}^{row} Fs_i$ $\quad Fst = 45.733 \cdot kips$

$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 29.902 \cdot kips \cdot ft$ $\quad Fc := 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$ $\quad Fc = 315.068 \cdot kips$

$Mn_1 := Mc + Mst$ $\quad Mn_1 = 398.525 \cdot kips \cdot in$ $\quad e_1 := \frac{Mn_1}{Pn_1}$ $\quad e_1 = 1.05 \cdot in$

$d_{row} = 7.125 \cdot in$

Find Pn and Mn at point of zero tension in bars:

$c := d_{row}$ $\quad c = 7.125 \cdot in$ $\quad a := \beta 1 \cdot c$ $\quad a = 6.056 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\qquad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\qquad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$i =$	$\varepsilon s_i =$	$fs_i =$ (ksi)	$Fs_i =$ (kips)	$Ms_i =$ (kips·in)	$A_i =$ (in^2)	$d_i =$ (in)
1	0.002	40	32.208	68.442	0.88	2.875
2	0	18.769	16.517	-35.099	0.88	7.125

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 33.343 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 487.24 \cdot kips \cdot in$

$Mn_2 := Mc + Mst$ $\quad Mn_2 = 520.584 \cdot kips \cdot in$ $\qquad Mn_2 = 521 \cdot kips \cdot in$

$Mn_1 = 398.525 \cdot kips \cdot in$ $\quad Mn_1 := if(Mn_1 < Mn_2, Mn_1, Mn_2)$ $\quad Mn_1 = 398.525 \cdot kips \cdot in$

$Pn_2 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_2 = 296 \cdot kips$ $\quad Pn_1 = 377.933 \cdot kips$ $\quad Pn_2 := if(Pn_2 < Pn_1, Pn_2, Pn_1)$ $\qquad Pn_2 = 295.82 \cdot kips$

Find Pn and Mn at point of 0.25 *fy tension in bars:

$$c := \frac{d_{row}}{1 + \frac{0.25 \cdot fy}{Es \cdot 0.003}} \qquad d_{row} = 7.125 \cdot in \qquad a := \beta 1 \cdot c$$

Find Moment:

$$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c} \qquad fs_i := \mathrm{if}(Es \cdot \varepsilon s_i > fy, fy, \mathrm{if}(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$$

$$Fs_i := fs_i \cdot A_i - \mathrm{if}(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$$

$d_i =$ (ft)	$\varepsilon s_i =$	$fs_i =$ (ksi)	$A_i =$ (in^2)	$Fs_i =$ (kips)	$Ms_i =$ (kips·in)
0.24	0.002	40	0.88	32.208	68.442
0.594	-0	-10	0.88	-8.8	18.7

$$Mst := \sum_{i=1}^{row} Ms_i \qquad Mst = 87 \cdot kips \cdot in \qquad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right) \qquad Mc = \blacksquare \cdot kips \cdot in \qquad Mn_3 := Mc + Mst \qquad Mn_3 = 593 \cdot kips \cdot in$$

$$Pn_3 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b \qquad Pn_3 = 245 \cdot kips$$

BY: KSM 12/16/2011

Find Pn and Mn at point of 0.50 *fy tension in bars:

$d_{row} = 7.125 \cdot in$ $\qquad c := \dfrac{d_{row}}{1 + \dfrac{0.50 \cdot fy}{Es \cdot 0.003}}$ $\qquad c = 5.793 \cdot in$ $\qquad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\qquad fs_i := \mathrm{if}(Es \cdot \varepsilon s_i > fy, fy, \mathrm{if}(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - \mathrm{if}(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\qquad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$ (ft)	$\varepsilon s_i =$	$fs_i =$ (ksi)	$A_i =$ (in^2)	$Fs_i =$ (kips)	$Ms_i =$ (kips·in)
0.24	0.002	40	0.88	32.208	68.442
0.594	-0.001	-20	0.88	-17.6	37.4

$Mst := \sum_{i=1}^{row} Ms_i$ $\qquad Mst = 106 \cdot kips \cdot in$ $\qquad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\qquad Mc = 509.883 \cdot kips \cdot in$ $\qquad Mn_4 := Mc + Mst$ $\qquad Mn_4 = 616 \cdot kips \cdot in$

$Pn_4 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\qquad Pn_4 = 216 \cdot kips$

Find Pn and Mn at Balanced:

$d_{row} = 7.125 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{fy}{Es \cdot 0.003}}$ $\quad c = 4.881 \cdot in$ $\quad a := \beta1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$ (ft)	$\varepsilon s_i =$	$fs_i =$ (ksi)	$A_i =$ (in^2)	$Fs_i =$ (kips)	$Ms_i =$ (kips·in)
0.24	0.001	35.754	0.88	28.472	60.503
0.594	-0.001	-40	0.88	-35.2	74.8

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 135 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 495.218 \cdot kips \cdot in$ $\quad Mn_5 := Mc + Mst$ $\quad Mn_5 = 631 \cdot kips \cdot in$

$Pn_5 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_5 = 163 \cdot kips$

Find Mn at ϕ Pn = 0.1*f'c*Ag:

$Pn_6 := \frac{0.1 \cdot f'c \cdot w \cdot h}{\phi}$ $\qquad Pn_6 = 48 \cdot kips$ $\qquad Pn_6 := if(Pn_6 > Pn_5, Pn_5, Pn_6)$ $\qquad Pn_6 = 48 \cdot kips$

Assume c=h/2 $\qquad c := h \cdot 0.5 \qquad c = 5 \cdot in$

Given

$$Pn_6 = \sum_{i=1}^{row} \left[\begin{array}{ll} (c - d_i) \cdot \frac{0.003}{c} \cdot Es \ldots + -if(d_i < \beta1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es < fy \\ fy - if(d_i < \beta1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \ge fy \\ -fy - if(d_i < \beta1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \le -fy \end{array} \right] \cdot A_i \ldots + 0.85 \cdot f'c \cdot \beta1 \cdot c \cdot b$$

$c := Find(c) \qquad c = 2.62 \cdot in$

$a := if(\beta1 \cdot c > h, h, \beta1 \cdot c) \qquad a = 2.22 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\qquad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\qquad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

d_i = (in)	εs_i =	fs_i = (psi)	A_i = (in^2)	Fs_i = (kips)	Ms_i = (in·kips)
2.875	-0	-8582.355	0.88	-7.552	-16.049
7.125	-0.005	-40000	0.88	-35.2	74.8

$Mst := \sum_{i=1}^{row} Ms_i$ $\qquad Mst = 59 \cdot kips \cdot in$

$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\qquad Mc = 352.831 \cdot kips \cdot in$

$Mn_6 := Mc + Mst$ $\qquad Mn_6 = 412 \cdot kips \cdot in$

$Pn_6 = 48 \cdot kips$

Find Mn at ϕ Pn = 0.0:

$Pn_7 := 0lb \qquad Pn_7 = 0lb$

Assume c=0.2h $\qquad c := 0.2 \cdot h$

Given

$$Pn_7 = \sum_{i=1}^{row} \left[\left[\begin{array}{ll} (c - d_i) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es < fy \\ + -\text{if}(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \\ fy - \text{if}(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \ge fy \\ -fy - \text{if}(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \le -fy \end{array} \right] \cdot A_i \right] \ldots$$
$$+ 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \qquad c = 1.993 \cdot in \qquad a := \text{if}(\beta 1 \cdot c > h, h, \beta 1 \cdot c) \qquad a = 1.694 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i)\cdot\dfrac{0.003}{c}$ $\qquad$ $fs_i := \text{if}(Es\cdot\varepsilon s_i > fy, fy, \text{if}(Es\cdot\varepsilon s_i < 0 - fy, 0 - fy, Es\cdot\varepsilon s_i))$

$Fs_i := fs_i\cdot A_i - \text{if}(\beta 1\cdot c > d_i, 0.85\cdot f'c, 0ksi)\cdot A_i$ $\qquad$ $Ms_i := Fs_i\cdot\left(\dfrac{h}{2} - d_i\right)$

$d_i = \begin{bmatrix} 0.24 \\ 0.594 \end{bmatrix} ft$ $\quad$ $\varepsilon s_i = \begin{bmatrix} -0.001 \\ -0.008 \end{bmatrix}$ $\quad$ $fs_i = \begin{bmatrix} -38.527 \\ -40 \end{bmatrix} \cdot ksi$ $\quad$ $A_i = \begin{bmatrix} 0.88 \\ 0.88 \end{bmatrix} \cdot in^2$ $\quad$ $Fs_i = \begin{bmatrix} -33.904 \\ -35.2 \end{bmatrix} \cdot kips$ $\quad$ $Ms_i = \begin{bmatrix} -72.045 \\ 74.8 \end{bmatrix} \cdot kips\cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$ $\qquad$ $Mst = 3\cdot kips\cdot in$

$Mc := 0.85\cdot f'c\cdot a\cdot w\cdot\left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\qquad$ $Mc = 287\cdot kips\cdot in$

$Mn_7 := Mc + Mst$ $\qquad$ $Mn_7 = 290\cdot kips\cdot in$

$j := 0..7$ $\phi_j := 1$

Results

$\phi_j \cdot Pn_j =$ (kips)	$\phi_j \cdot Mn_j =$ (kips·in)
377.933	0
377.933	398.525
295.82	520.584
245.029	593.337
215.517	615.725
162.542	630.521
48	411.582
0	289.752

Criteria:

Depth	$h = 10 \cdot in$	$bar = 6$
Width	$w = 12 \cdot in$	$tie = 2$
Eff. Width	$b = 12 \cdot in$	$fy = 40 \cdot ksi$

$cover = 2.25 \cdot in$ $f'c = 4000 \cdot psi$

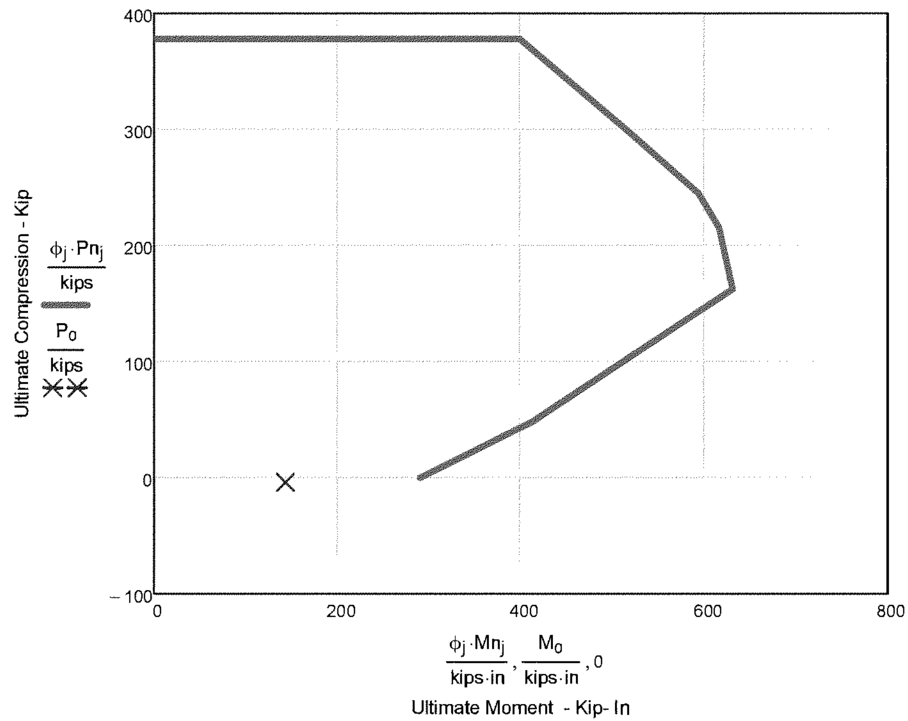
$Ag = 120 \cdot in^2$ $Ast = 1.76 \cdot in^2$

$\frac{Ast}{w \cdot h} = 0.0147$ Actual ratio ρ

$\frac{Ast}{Ag} = 0.0147$ Eff. ratio ρ

Demand Loads Water to Top of Wall:

Axial Load from CPGA output: $P_0 := -3.9\text{kips}$ Moment from CPGA output: $M_0 := 143\text{kips}\cdot\text{in}$



Interaction Diagram

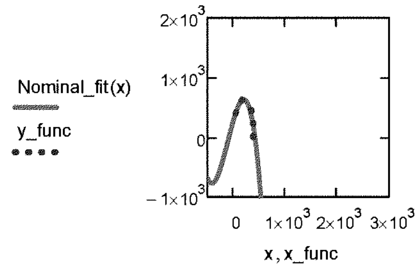
Nominal Curve Used to Find Factor of Safety:

Data Points set to function: $y\_func := \frac{Mn}{kips \cdot in}$ $x\_func := \frac{Pn}{kips}$

Degree of Polynominal Fit: $k := 3$

Returns a vector which interp uses to find the kth order polynomial that best fits the x and y data values

$Nominal := regress(x\_func, y\_func, k)$ $Nominal\_fit(x) := interp(Nominal, x\_func, y\_func, x)$



Strength Factor of Safety:

Read the Corresponding Moment for the demand axial load (
$P_0 = -3.9 \cdot kips$)

$M_{capacity} := Nominal\_fit\left(\frac{P_0}{kips}\right)$ $M_{capacity} = 274.818$

Demand Moment: $M_{demand} := \frac{M_0}{kips \cdot in}$ $M_{demand} = 143$

Factor of Safety: $FS_{strength} := \frac{M_{capacity}}{M_{demand}}$ $FS_{strength} = 1.9$

BY: KSM 12/16/2011

FEMA 310 GUIDANCE

4.2.4.4 Component Strength

Component strength for all actions shall be taken as the expected strength, Q_{ce}. Unless calculated otherwise, the expected strength shall be assumed equal to the nominal strength multiplied by 1.25. Alternatively, if allowable stresses are used, nominal strengths shall be taken as the allowable values multiplied by the following values:

Steel	1.7
Masonry	2.5
Wood	2.0

Except for wood diaphragms and wood and masonry shear walls, the allowable values shall not include a one-third increase for short term loading.

When calculating capacities of deteriorated elements, the evaluating design professional shall make

US Army Corps of Engineers

Probability of Failure
CIDMO Type R floodwall pile footings 18" Pile capacity

Comp by:KSM 12-8-11
Chkd by:

I. *Objective*

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. *References*

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. *Situation*

1. This structure does not meet the allowable compression loads for which it has been determined 99.8% reliability can be assigned. See mathcad sheets for calculations.
2. EM 1110-2-561 states that the coefficient of variation (COV) are the following:
   - Compression Capacity 25%
   - Tension Capacity 18%
3. Values used for original capacity check:
   - Allowable Compression Capacity $P_{callow} := 37.17kip$
   - Allowable Tension Capacity $P_{tallow} := 20.25kip$

IV. *Variable Definitions*

FS_M = Factor of Safety under mean material parameters

FS_u = Factor of Safety due to upper bound value of the ULTIMATE PILE COMPRESSION CAPACITY

FS_l = Factor of Safety due to lower bound value of the ULTIMATE PILE COMPRESSION CAPACITY

ΔF_C = Difference in Factors of Safety due to change in ULTIMATE PILE COMPRESSION CAPACITY

σ_F = Standard Deviation of the Factor of Safety

V_F = Coefficient of Variation of the Factor of Safety

β_{LN} = Lognormal Reliability Index

R = Reliability

P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. *Calculating Factors of Safety*

WATER AT TOP OF WALL

18-inch tapered piles

Actual Compression on Piles

$P_u := 42.8kip$

Ultimate Compression Capacity

$P_{cult} := 63.184kip$

$P_{cupper} := P_{cult} \cdot 1.25 = 78.98 \cdot kip$

$P_{clower} := P_{cult} \cdot 0.75 = 47.388 \cdot kip$

Mean Factor of Safety

$FS_M := \frac{P_{cult}}{P_u}$ $\quad FS_M = 1.476$

Upper Axial Compression Capacity

$FS_u := \frac{P_{cupper}}{P_u}$ $\quad FS_u = 1.845$

Lower Axial Compression Capacity

$FS_l := \frac{P_{clower}}{P_u}$ $\quad FS_l = 1.107$

VI. *Probability of Failure Calculation*

$\Delta F_C := FS_u - FS_l$ $\qquad\qquad$ $\Delta F_C = 0.738$

$\sigma_F := \sqrt{\left(\frac{\Delta F_C}{2}\right)^2}$ $\qquad\qquad$ $\sigma_F = 0.369$

$V_F := \frac{\sigma_F}{FS_M}$ $\qquad\qquad$ $V_F = 0.25$

$\beta_{LN} := \frac{\ln\left(\frac{FS_M}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\qquad\qquad$ $\beta_{LN} = 1.459$

$R := \mathrm{cnorm}(\beta_{LN})$ $\qquad\qquad$ $R = 92.77 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad\qquad$ $P_F = 7.23 \cdot \%$

V. *Calculating Factors of Safety*

WATER AT 1-FOOT DOWN FROM TOP OF WALL

18-inch tapered piles

Actual Compression on Piles

$P_u := 38\text{kip}$

Ultimate Compression Capacity

$P_{cupper} := P_{cult} \cdot 1.25 = 78.98 \cdot \text{kip}$

$P_{clower} := P_{cult} \cdot 0.75 = 47.388 \cdot \text{kip}$

Mean Factor of Safety

$FS_M := \dfrac{P_{cult}}{P_u}$ $\qquad FS_M = 1.663$

Upper Axial Compression Capacity

$FS_u := \dfrac{P_{cupper}}{P_u}$ $\qquad FS_u = 2.078$

Lower Axial Compression Capacity

$FS_l := \dfrac{P_{clower}}{P_u}$ $\qquad FS_l = 1.247$

VI. *Probability of Failure Calculation*

$\Delta F_C := FS_u - FS_l$ $\qquad$ $\Delta F_C = 0.831$

$\sigma_F := \sqrt{\left(\frac{\Delta F_C}{2}\right)^2}$ $\qquad$ $\sigma_F = 0.416$

$V_F := \frac{\sigma_F}{FS_M}$ $\qquad$ $V_F = 0.25$

$\beta_{LN} := \frac{\ln\left(\frac{FS_M}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\qquad$ $\beta_{LN} = 1.942$

$R := \text{cnorm}\left(\beta_{LN}\right)$ $\qquad$ $R = 97.39 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad$ $P_F = 2.61 \cdot \%$

V. *Calculating Factors of Safety*

WATER AT 2-FOOT DOWN FROM TOP OF WALL

18-inch tapered piles

Actual Compression on Piles

$P_u := 33.6\text{kip}$

Ultimate Compression Capacity

$P_{cupper} := P_{cult} \cdot 1.25 = 78.98 \cdot \text{kip}$

$P_{clower} := P_{cult} \cdot 0.75 = 47.388 \cdot \text{kip}$

Mean Factor of Safety

$FS_M := \dfrac{P_{cult}}{P_u}$ $\qquad FS_M = 1.88$

Upper Axial Compression Capacity

$FS_u := \dfrac{P_{cupper}}{P_u}$ $\qquad FS_u = 2.351$

Lower Axial Compression Capacity

$FS_l := \dfrac{P_{clower}}{P_u}$ $\qquad FS_l = 1.41$

VI. Probability of Failure Calculation

$\Delta F_C := FS_u - FS_l$ $\Delta F_C = 0.94$

$\sigma_F := \sqrt{\left(\frac{\Delta F_C}{2}\right)^2}$ $\sigma_F = 0.47$

$V_F := \frac{\sigma_F}{FS_M}$ $V_F = 0.25$

$\beta_{LN} := \frac{\ln\left(\frac{FS_M}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\beta_{LN} = 2.442$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 99.27 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 0.73 \cdot \%$

```
CPGA - CASE PILE GROUP ANALYSIS PROGRAM
RUN DATE: 07-DEC-2011   RUN TIME: 14.42.38


    FOR PILES WITH UNSUPPORTED HEIGHT:
         A. CPGA CANNOT CALCULATE PMAXMOM FOR NH TYPE SOIL
         B. THE ALLOWABLE STRESS CHECKS, ASC AND AST, ARE
            NOT FULLY DEVELOPED FOR UNSUPPORTED PILES.
            WORK IS IN PROGRESS TO COMPLETE THIS ASPECT OF CPGA.

    ELASTIC CENTER LOCATION IS NOT COMPUTED FOR 3-DIMENSIONAL PROBLEMS.


CID-MO SECTION TYPE R
DATA UNKNOWN - REJECTED.


THERE ARE  56 PILES AND
            1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                           X           Y           Z
                                         -----       -----       -----
WITH DIAGONAL COORDINATES = (          -20.50 ,      2.00 ,       .00 )
                            (           20.50 ,     11.50 ,       .00 )

******************************************************************************

         PILE PROPERTIES AS INPUT


      E            I1            I2            A            C33           B66
     KSI          IN**4         IN**4         IN**2
  .36050E+04    .42180E+04    .42180E+04    .22500E+03    .20000E+01    .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


      E            I1            I2            A            C33           B66
     KSI          IN**4         IN**4         IN**2
  .36050E+04    .10000E+04    .14400E+04    .12000E+03    .20000E+01    .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56

******************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   NH      ESOIL       LENGTH         L             LU
         K/IN**3                     FT             FT
         .60000E-01      L       .20000E+02    .00000E+00

  ESOIL(ORIGINAL)      RGROUP     RCYCLIC
    K/IN**3
    .60000E-01         .1000E+01  .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


   NH      ESOIL       LENGTH         L             LU
         K/IN**3                     FT             FT
         .60000E-01      L       .15000E+02    .00000E+00

  ESOIL(ORIGINAL)      RGROUP     RCYCLIC
    K/IN**3
    .60000E-01         .1000E+01  .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56

******************************************************************************
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PILE STIFFNESSES AS CALCULATED FROM PROPERTIES

.14829E+03	.00000E+00	.00000E+00	.00000E+00	.66141E+04	.00000E+00
.00000E+00	.14829E+03	.00000E+00	-.66141E+04	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.67594E+04	.00000E+00	.00000E+00	.00000E+00
.00000E+00	-.66141E+04	.00000E+00	.47570E+06	.00000E+00	.00000E+00
.66141E+04	.00000E+00	.00000E+00	.00000E+00	.47570E+06	.00000E+00
.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00

THIS MATRIX APPLIES TO THE FOLLOWING PILES -

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LENGTH LESS THAN 5T2 FOR PILE 29

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LENGTH LESS THAN 5T2 FOR PILE    46
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PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	-18.00	2.00	.00	V	.00	20.00	F
2	-12.00	2.00	.00	V	.00	20.00	F
3	-6.00	2.00	.00	V	.00	20.00	F
4	.00	2.00	.00	V	.00	20.00	F
5	6.00	2.00	.00	V	.00	20.00	F
6	12.00	2.00	.00	V	.00	20.00	F
7	18.00	2.00	.00	V	.00	20.00	F
8	-18.00	6.00	.00	V	.00	20.00	F
9	-12.00	6.00	.00	V	.00	20.00	F
10	-6.00	6.00	.00	V	.00	20.00	F
11	.00	6.00	.00	V	.00	20.00	F
12	6.00	6.00	.00	V	.00	20.00	F
13	12.00	6.00	.00	V	.00	20.00	F
14	18.00	6.00	.00	V	.00	20.00	F
15	-20.50	11.50	.00	V	.00	15.00	F
16	-19.50	11.50	.00	V	.00	15.00	F
17	-18.50	11.50	.00	V	.00	15.00	F
18	-17.50	11.50	.00	V	.00	15.00	F
19	-16.50	11.50	.00	V	.00	15.00	F
20	-15.50	11.50	.00	V	.00	15.00	F
21	-14.50	11.50	.00	V	.00	15.00	F
22	-13.50	11.50	.00	V	.00	15.00	F
23	-12.50	11.50	.00	V	.00	15.00	F
24	-11.50	11.50	.00	V	.00	15.00	F
25	-10.50	11.50	.00	V	.00	15.00	F
26	-9.50	11.50	.00	V	.00	15.00	F
27	-8.50	11.50	.00	V	.00	15.00	F
28	-7.50	11.50	.00	V	.00	15.00	F
29	-6.50	11.50	.00	V	.00	15.00	F
30	-5.50	11.50	.00	V	.00	15.00	F

31	-4.50	11.50	.00	V	.00	15.00	F
32	-3.50	11.50	.00	V	.00	15.00	F
33	-2.50	11.50	.00	V	.00	15.00	F
34	-1.50	11.50	.00	V	.00	15.00	F
35	-.50	11.50	.00	V	.00	15.00	F
36	.50	11.50	.00	V	.00	15.00	F
37	1.50	11.50	.00	V	.00	15.00	F
38	2.50	11.50	.00	V	.00	15.00	F
39	3.50	11.50	.00	V	.00	15.00	F
40	4.50	11.50	.00	V	.00	15.00	F
41	5.50	11.50	.00	V	.00	15.00	F
42	6.50	11.50	.00	V	.00	15.00	F
43	7.50	11.50	.00	V	.00	15.00	F
44	8.50	11.50	.00	V	.00	15.00	F
45	9.50	11.50	.00	V	.00	15.00	F
46	10.50	11.50	.00	V	.00	15.00	F
47	11.50	11.50	.00	V	.00	15.00	F
48	12.50	11.50	.00	V	.00	15.00	F
49	13.50	11.50	.00	V	.00	15.00	F
50	14.50	11.50	.00	V	.00	15.00	F
51	15.50	11.50	.00	V	.00	15.00	F
52	16.50	11.50	.00	V	.00	15.00	F
53	17.50	11.50	.00	V	.00	15.00	F
54	18.50	11.50	.00	V	.00	15.00	F
55	19.50	11.50	.00	V	.00	15.00	F
56	20.50	11.50	.00	V	.00	15.00	F

						910.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-291.7	295.4	571.7	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.61279E+04	.00000E+00	.00000E+00	.00000E+00	.23837E+06	-.65881E+06
.00000E+00	.55780E+04	.00000E+00	-.20972E+06	.00000E+00	-.50932E-10
.00000E+00	.00000E+00	.29651E+06	.32402E+08	.00000E+00	.00000E+00
.00000E+00	-.20972E+06	.32402E+08	.41301E+10	.00000E+00	-.34925E-09
.23837E+06	.00000E+00	.00000E+00	.00000E+00	.62484E+10	-.24561E+08
-.65881E+06	-.50932E-10	.00000E+00	-.34925E-09	-.24561E+08	.20028E+09

56 PILES 1 LOAD CASES

LOAD CASE 1. NUMBER OF FAILURES = 56. NUMBER OF PILES IN TENSION = 42.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	-.2329E-17	-.5464E-01	.7833E-02	-.6257E-04	.3701E-23	-.2167E-19

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
2	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*

3	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
4	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
5	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
6	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
7	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
8	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
9	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
10	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
11	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
12	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
13	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
14	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
15	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
16	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
17	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
18	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
19	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
20	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
21	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
22	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
23	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
24	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
25	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
26	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
27	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
28	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
29	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
30	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
31	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
32	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
33	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
34	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
35	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
36	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
37	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
38	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
39	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
40	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
41	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
42	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
43	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
44	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
45	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
46	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
47	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
48	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
49	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
50	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
51	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
52	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
53	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
54	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
55	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
56	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-7.7	42.8	331.6	.0	.0
2	.0	-7.7	42.8	331.6	.0	.0
3	.0	-7.7	42.8	331.6	.0	.0
4	.0	-7.7	42.8	331.6	.0	.0
5	.0	-7.7	42.8	331.6	.0	.0
6	.0	-7.7	42.8	331.6	.0	.0
7	.0	-7.7	42.8	331.6	.0	.0
8	.0	-7.7	22.5	331.6	.0	.0
9	.0	-7.7	22.5	331.6	.0	.0
10	.0	-7.7	22.5	331.6	.0	.0
11	.0	-7.7	22.5	331.6	.0	.0
12	.0	-7.7	22.5	331.6	.0	.0
13	.0	-7.7	22.5	331.6	.0	.0
14	.0	-7.7	22.5	331.6	.0	.0
15	.0	-4.4	-3.9	143.0	.0	.0
16	.0	-4.4	-3.9	143.0	.0	.0
17	.0	-4.4	-3.9	143.0	.0	.0
18	.0	-4.4	-3.9	143.0	.0	.0
19	.0	-4.4	-3.9	143.0	.0	.0
20	.0	-4.4	-3.9	143.0	.0	.0
21	.0	-4.4	-3.9	143.0	.0	.0

22	.0	-4.4	-3.9	143.0	.0	.0
23	.0	-4.4	-3.9	143.0	.0	.0
24	.0	-4.4	-3.9	143.0	.0	.0
25	.0	-4.4	-3.9	143.0	.0	.0
26	.0	-4.4	-3.9	143.0	.0	.0
27	.0	-4.4	-3.9	143.0	.0	.0
28	.0	-4.4	-3.9	143.0	.0	.0
29	.0	-4.4	-3.9	143.0	.0	.0
30	.0	-4.4	-3.9	143.0	.0	.0
31	.0	-4.4	-3.9	143.0	.0	.0
32	.0	-4.4	-3.9	143.0	.0	.0
33	.0	-4.4	-3.9	143.0	.0	.0
34	.0	-4.4	-3.9	143.0	.0	.0
35	.0	-4.4	-3.9	143.0	.0	.0
36	.0	-4.4	-3.9	143.0	.0	.0
37	.0	-4.4	-3.9	143.0	.0	.0
38	.0	-4.4	-3.9	143.0	.0	.0
39	.0	-4.4	-3.9	143.0	.0	.0
40	.0	-4.4	-3.9	143.0	.0	.0
41	.0	-4.4	-3.9	143.0	.0	.0
42	.0	-4.4	-3.9	143.0	.0	.0
43	.0	-4.4	-3.9	143.0	.0	.0
44	.0	-4.4	-3.9	143.0	.0	.0
45	.0	-4.4	-3.9	143.0	.0	.0
46	.0	-4.4	-3.9	143.0	.0	.0
47	.0	-4.4	-3.9	143.0	.0	.0
48	.0	-4.4	-3.9	143.0	.0	.0
49	.0	-4.4	-3.9	143.0	.0	.0
50	.0	-4.4	-3.9	143.0	.0	.0
51	.0	-4.4	-3.9	143.0	.0	.0
52	.0	-4.4	-3.9	143.0	.0	.0
53	.0	-4.4	-3.9	143.0	.0	.0
54	.0	-4.4	-3.9	143.0	.0	.0
55	.0	-4.4	-3.9	143.0	.0	.0
56	.0	-4.4	-3.9	143.0	.0	.0

NO FILES WERE GENERATED DURING THIS RUN.
Stop - Program terminated.

```
CPGA - CASE PILE GROUP ANALYSIS PROGRAM
RUN DATE: 08-DEC-2011   RUN TIME: 21.02.37



    FOR PILES WITH UNSUPPORTED HEIGHT:
         A. CPGA CANNOT CALCULATE PMAXMOM FOR NH TYPE SOIL
         B. THE ALLOWABLE STRESS CHECKS, ASC AND AST, ARE
            NOT FULLY DEVELOPED FOR UNSUPPORTED PILES.
            WORK IS IN PROGRESS TO COMPLETE THIS ASPECT OF CPGA.

    ELASTIC CENTER LOCATION IS NOT COMPUTED FOR 3-DIMENSIONAL PROBLEMS.


CID-MO SECTION TYPE R
DATA UNKNOWN - REJECTED.



THERE ARE  56 PILES AND
            1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                       X          Y          Z
                                     -----      -----      -----
WITH DIAGONAL COORDINATES = (      -20.50 ,     2.00 ,      .00 )
                            (       20.50 ,    11.50 ,      .00 )

********************************************************************************

         PILE PROPERTIES AS INPUT


      E            I1            I2            A            C33          B66
     KSI          IN**4         IN**4         IN**2
  .36050E+04    .42180E+04    .42180E+04    .22500E+03    .20000E+01    .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

    1    2    3    4    5    6    7    8    9   10   11   12   13   14


      E            I1            I2            A            C33          B66
     KSI          IN**4         IN**4         IN**2
  .36050E+04    .10000E+04    .14400E+04    .12000E+03    .20000E+01    .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
   31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
   47   48   49   50   51   52   53   54   55   56


********************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   NH      ESOIL       LENGTH         L             LU
          K/IN**3                     FT            FT
          .60000E-01     L        .20000E+02     .00000E+00

 ESOIL(ORIGINAL)       RGROUP      RCYCLIC
   K/IN**3
   .60000E-01          .1000E+01   .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

    1    2    3    4    5    6    7    8    9   10   11   12   13   14


   NH      ESOIL       LENGTH         L             LU
          K/IN**3                     FT            FT
          .60000E-01     L        .15000E+02     .00000E+00

 ESOIL(ORIGINAL)       RGROUP      RCYCLIC
   K/IN**3
   .60000E-01          .1000E+01   .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
   31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
   47   48   49   50   51   52   53   54   55   56


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PILE STIFFNESSES AS CALCULATED FROM PROPERTIES

.14829E+03	.00000E+00	.00000E+00	.00000E+00	.66141E+04	.00000E+00
.00000E+00	.14829E+03	.00000E+00	-.66141E+04	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.67594E+04	.00000E+00	.00000E+00	.00000E+00
.00000E+00	-.66141E+04	.00000E+00	.47570E+06	.00000E+00	.00000E+00
.66141E+04	.00000E+00	.00000E+00	.00000E+00	.47570E+06	.00000E+00
.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00

THIS MATRIX APPLIES TO THE FOLLOWING PILES -

1

* *
LENGTH LESS THAN 5T2 FOR PILE 15
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 16
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 17
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 18
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 19
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 20
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 21
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 22
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 23
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 24
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 25
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 26
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* *
LENGTH LESS THAN 5T2 FOR PILE 27
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* *
LENGTH LESS THAN 5T2 FOR PILE 28
* *

* *
LENGTH LESS THAN 5T2 FOR PILE 29

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LENGTH LESS THAN 5T2 FOR PILE    30
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LENGTH LESS THAN 5T2 FOR PILE    31
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LENGTH LESS THAN 5T2 FOR PILE    32
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LENGTH LESS THAN 5T2 FOR PILE    33
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LENGTH LESS THAN 5T2 FOR PILE    34
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LENGTH LESS THAN 5T2 FOR PILE    35
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LENGTH LESS THAN 5T2 FOR PILE    36
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LENGTH LESS THAN 5T2 FOR PILE    37
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LENGTH LESS THAN 5T2 FOR PILE    38
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LENGTH LESS THAN 5T2 FOR PILE    39
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LENGTH LESS THAN 5T2 FOR PILE    40
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LENGTH LESS THAN 5T2 FOR PILE    41
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LENGTH LESS THAN 5T2 FOR PILE    42
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LENGTH LESS THAN 5T2 FOR PILE    43
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LENGTH LESS THAN 5T2 FOR PILE    44
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LENGTH LESS THAN 5T2 FOR PILE    45
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LENGTH LESS THAN 5T2 FOR PILE    46
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LENGTH LESS THAN 5T2 FOR PILE   47
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LENGTH LESS THAN 5T2 FOR PILE   48
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LENGTH LESS THAN 5T2 FOR PILE   49
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LENGTH LESS THAN 5T2 FOR PILE   50
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LENGTH LESS THAN 5T2 FOR PILE   51
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LENGTH LESS THAN 5T2 FOR PILE   52
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LENGTH LESS THAN 5T2 FOR PILE   53
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LENGTH LESS THAN 5T2 FOR PILE   54
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LENGTH LESS THAN 5T2 FOR PILE   55
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LENGTH LESS THAN 5T2 FOR PILE   56
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PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	-18.00	2.00	.00	V	.00	20.00	F
2	-12.00	2.00	.00	V	.00	20.00	F
3	-6.00	2.00	.00	V	.00	20.00	F
4	.00	2.00	.00	V	.00	20.00	F
5	6.00	2.00	.00	V	.00	20.00	F
6	12.00	2.00	.00	V	.00	20.00	F
7	18.00	2.00	.00	V	.00	20.00	F
8	-18.00	6.00	.00	V	.00	20.00	F
9	-12.00	6.00	.00	V	.00	20.00	F
10	-6.00	6.00	.00	V	.00	20.00	F
11	.00	6.00	.00	V	.00	20.00	F
12	6.00	6.00	.00	V	.00	20.00	F
13	12.00	6.00	.00	V	.00	20.00	F
14	18.00	6.00	.00	V	.00	20.00	F
15	-20.50	11.50	.00	V	.00	15.00	F
16	-19.50	11.50	.00	V	.00	15.00	F
17	-18.50	11.50	.00	V	.00	15.00	F
18	-17.50	11.50	.00	V	.00	15.00	F
19	-16.50	11.50	.00	V	.00	15.00	F
20	-15.50	11.50	.00	V	.00	15.00	F
21	-14.50	11.50	.00	V	.00	15.00	F
22	-13.50	11.50	.00	V	.00	15.00	F
23	-12.50	11.50	.00	V	.00	15.00	F
24	-11.50	11.50	.00	V	.00	15.00	F
25	-10.50	11.50	.00	V	.00	15.00	F
26	-9.50	11.50	.00	V	.00	15.00	F
27	-8.50	11.50	.00	V	.00	15.00	F
28	-7.50	11.50	.00	V	.00	15.00	F
29	-6.50	11.50	.00	V	.00	15.00	F
30	-5.50	11.50	.00	V	.00	15.00	F

31	-4.50	11.50	.00	V	.00	15.00	F
32	-3.50	11.50	.00	V	.00	15.00	F
33	-2.50	11.50	.00	V	.00	15.00	F
34	-1.50	11.50	.00	V	.00	15.00	F
35	-.50	11.50	.00	V	.00	15.00	F
36	.50	11.50	.00	V	.00	15.00	F
37	1.50	11.50	.00	V	.00	15.00	F
38	2.50	11.50	.00	V	.00	15.00	F
39	3.50	11.50	.00	V	.00	15.00	F
40	4.50	11.50	.00	V	.00	15.00	F
41	5.50	11.50	.00	V	.00	15.00	F
42	6.50	11.50	.00	V	.00	15.00	F
43	7.50	11.50	.00	V	.00	15.00	F
44	8.50	11.50	.00	V	.00	15.00	F
45	9.50	11.50	.00	V	.00	15.00	F
46	10.50	11.50	.00	V	.00	15.00	F
47	11.50	11.50	.00	V	.00	15.00	F
48	12.50	11.50	.00	V	.00	15.00	F
49	13.50	11.50	.00	V	.00	15.00	F
50	14.50	11.50	.00	V	.00	15.00	F
51	15.50	11.50	.00	V	.00	15.00	F
52	16.50	11.50	.00	V	.00	15.00	F
53	17.50	11.50	.00	V	.00	15.00	F
54	18.50	11.50	.00	V	.00	15.00	F
55	19.50	11.50	.00	V	.00	15.00	F
56	20.50	11.50	.00	V	.00	15.00	F

						910.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-251.0	288.2	772.9	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.61279E+04	.00000E+00	.00000E+00	.00000E+00	.23837E+06	-.65881E+06
.00000E+00	.55780E+04	.00000E+00	-.20972E+06	.00000E+00	-.50932E-10
.00000E+00	.00000E+00	.29651E+06	.32402E+08	.00000E+00	.00000E+00
.00000E+00	-.20972E+06	.32402E+08	.41301E+10	.00000E+00	-.34925E-09
.23837E+06	.00000E+00	.00000E+00	.00000E+00	.62484E+10	-.24561E+08
-.65881E+06	-.50932E-10	.00000E+00	-.34925E-09	-.24561E+08	.20028E+09

56 PILES 1 LOAD CASES

LOAD CASE 1. NUMBER OF FAILURES = 56. NUMBER OF PILES IN TENSION = 42.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	-.2006E-17	-.4705E-01	.6922E-02	-.5445E-04	.3187E-23	-.1866E-19

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
2	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*

3	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
4	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
5	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
6	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
7	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
8	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
9	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
10	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
11	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
12	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
13	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
14	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
15	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
16	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
17	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
18	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
19	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
20	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
21	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
22	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
23	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
24	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
25	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
26	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
27	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
28	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
29	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
30	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
31	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
32	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
33	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
34	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
35	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
36	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
37	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
38	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
39	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
40	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
41	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
42	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
43	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
44	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
45	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
46	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
47	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
48	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
49	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
50	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
51	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
52	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
53	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
54	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
55	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
56	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-6.6	38.0	285.3	.0	.0
2	.0	-6.6	38.0	285.3	.0	.0
3	.0	-6.6	38.0	285.3	.0	.0
4	.0	-6.6	38.0	285.3	.0	.0
5	.0	-6.6	38.0	285.3	.0	.0
6	.0	-6.6	38.0	285.3	.0	.0
7	.0	-6.6	38.0	285.3	.0	.0
8	.0	-6.6	20.3	285.3	.0	.0
9	.0	-6.6	20.3	285.3	.0	.0
10	.0	-6.6	20.3	285.3	.0	.0
11	.0	-6.6	20.3	285.3	.0	.0
12	.0	-6.6	20.3	285.3	.0	.0
13	.0	-6.6	20.3	285.3	.0	.0
14	.0	-6.6	20.3	285.3	.0	.0
15	.0	-3.8	-2.8	123.0	.0	.0
16	.0	-3.8	-2.8	123.0	.0	.0
17	.0	-3.8	-2.8	123.0	.0	.0
18	.0	-3.8	-2.8	123.0	.0	.0
19	.0	-3.8	-2.8	123.0	.0	.0
20	.0	-3.8	-2.8	123.0	.0	.0
21	.0	-3.8	-2.8	123.0	.0	.0

22	.0	-3.8	-2.8	123.0	.0	.0
23	.0	-3.8	-2.8	123.0	.0	.0
24	.0	-3.8	-2.8	123.0	.0	.0
25	.0	-3.8	-2.8	123.0	.0	.0
26	.0	-3.8	-2.8	123.0	.0	.0
27	.0	-3.8	-2.8	123.0	.0	.0
28	.0	-3.8	-2.8	123.0	.0	.0
29	.0	-3.8	-2.8	123.0	.0	.0
30	.0	-3.8	-2.8	123.0	.0	.0
31	.0	-3.8	-2.8	123.0	.0	.0
32	.0	-3.8	-2.8	123.0	.0	.0
33	.0	-3.8	-2.8	123.0	.0	.0
34	.0	-3.8	-2.8	123.0	.0	.0
35	.0	-3.8	-2.8	123.0	.0	.0
36	.0	-3.8	-2.8	123.0	.0	.0
37	.0	-3.8	-2.8	123.0	.0	.0
38	.0	-3.8	-2.8	123.0	.0	.0
39	.0	-3.8	-2.8	123.0	.0	.0
40	.0	-3.8	-2.8	123.0	.0	.0
41	.0	-3.8	-2.8	123.0	.0	.0
42	.0	-3.8	-2.8	123.0	.0	.0
43	.0	-3.8	-2.8	123.0	.0	.0
44	.0	-3.8	-2.8	123.0	.0	.0
45	.0	-3.8	-2.8	123.0	.0	.0
46	.0	-3.8	-2.8	123.0	.0	.0
47	.0	-3.8	-2.8	123.0	.0	.0
48	.0	-3.8	-2.8	123.0	.0	.0
49	.0	-3.8	-2.8	123.0	.0	.0
50	.0	-3.8	-2.8	123.0	.0	.0
51	.0	-3.8	-2.8	123.0	.0	.0
52	.0	-3.8	-2.8	123.0	.0	.0
53	.0	-3.8	-2.8	123.0	.0	.0
54	.0	-3.8	-2.8	123.0	.0	.0
55	.0	-3.8	-2.8	123.0	.0	.0
56	.0	-3.8	-2.8	123.0	.0	.0

NO FILES WERE GENERATED DURING THIS RUN.
Stop - Program terminated.

```
CPGA - CASE PILE GROUP ANALYSIS PROGRAM
RUN DATE: 08-DEC-2011   RUN TIME: 21.03.50


    FOR PILES WITH UNSUPPORTED HEIGHT:
         A. CPGA CANNOT CALCULATE PMAXMOM FOR NH TYPE SOIL
         B. THE ALLOWABLE STRESS CHECKS, ASC AND AST, ARE
            NOT FULLY DEVELOPED FOR UNSUPPORTED PILES.
            WORK IS IN PROGRESS TO COMPLETE THIS ASPECT OF CPGA.

    ELASTIC CENTER LOCATION IS NOT COMPUTED FOR 3-DIMENSIONAL PROBLEMS.


CID-MO SECTION TYPE R
DATA UNKNOWN - REJECTED.


THERE ARE  56 PILES AND
            1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                       X          Y          Z
                                     -----      -----      -----
WITH DIAGONAL COORDINATES = (      -20.50 ,     2.00 ,      .00 )
                            (       20.50 ,    11.50 ,      .00 )

******************************************************************************

         PILE PROPERTIES AS INPUT


      E            I1            I2            A            C33           B66
     KSI          IN**4         IN**4         IN**2
  .36050E+04    .42180E+04    .42180E+04    .22500E+03    .20000E+01    .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


      E            I1            I2            A            C33           B66
     KSI          IN**4         IN**4         IN**2
  .36050E+04    .10000E+04    .14400E+04    .12000E+03    .20000E+01    .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56

******************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   NH      ESOIL       LENGTH          L            LU
          K/IN**3                      FT           FT
          .60000E-01     L          .20000E+02    .00000E+00

 ESOIL(ORIGINAL)      RGROUP      RCYCLIC
   K/IN**3
  .60000E-01          .1000E+01   .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


   NH      ESOIL       LENGTH          L            LU
          K/IN**3                      FT           FT
          .60000E-01     L          .15000E+02    .00000E+00

 ESOIL(ORIGINAL)      RGROUP      RCYCLIC
   K/IN**3
  .60000E-01          .1000E+01   .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56

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PILE STIFFNESSES AS CALCULATED FROM PROPERTIES

.14829E+03	.00000E+00	.00000E+00	.00000E+00	.66141E+04	.00000E+00
.00000E+00	.14829E+03	.00000E+00	-.66141E+04	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.67594E+04	.00000E+00	.00000E+00	.00000E+00
.00000E+00	-.66141E+04	.00000E+00	.47570E+06	.00000E+00	.00000E+00
.66141E+04	.00000E+00	.00000E+00	.00000E+00	.47570E+06	.00000E+00
.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00

THIS MATRIX APPLIES TO THE FOLLOWING PILES -

1

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LENGTH LESS THAN 5T2 FOR PILE   15
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LENGTH LESS THAN 5T2 FOR PILE   16
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LENGTH LESS THAN 5T2 FOR PILE   17
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LENGTH LESS THAN 5T2 FOR PILE   18
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LENGTH LESS THAN 5T2 FOR PILE   19
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LENGTH LESS THAN 5T2 FOR PILE   20
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LENGTH LESS THAN 5T2 FOR PILE   21
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LENGTH LESS THAN 5T2 FOR PILE   22
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LENGTH LESS THAN 5T2 FOR PILE   23
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LENGTH LESS THAN 5T2 FOR PILE   24
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LENGTH LESS THAN 5T2 FOR PILE   25
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LENGTH LESS THAN 5T2 FOR PILE   26
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LENGTH LESS THAN 5T2 FOR PILE   27
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LENGTH LESS THAN 5T2 FOR PILE   28
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LENGTH LESS THAN 5T2 FOR PILE   29
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LENGTH LESS THAN 5T2 FOR PILE    30
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LENGTH LESS THAN 5T2 FOR PILE    31
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LENGTH LESS THAN 5T2 FOR PILE    32
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LENGTH LESS THAN 5T2 FOR PILE    33
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LENGTH LESS THAN 5T2 FOR PILE    34
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LENGTH LESS THAN 5T2 FOR PILE    35
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LENGTH LESS THAN 5T2 FOR PILE    36
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LENGTH LESS THAN 5T2 FOR PILE    37
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LENGTH LESS THAN 5T2 FOR PILE    38
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LENGTH LESS THAN 5T2 FOR PILE    39
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LENGTH LESS THAN 5T2 FOR PILE    40
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LENGTH LESS THAN 5T2 FOR PILE    41
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LENGTH LESS THAN 5T2 FOR PILE    42
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LENGTH LESS THAN 5T2 FOR PILE    43
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LENGTH LESS THAN 5T2 FOR PILE    44
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LENGTH LESS THAN 5T2 FOR PILE    45
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LENGTH LESS THAN 5T2 FOR PILE    46
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LENGTH LESS THAN 5T2 FOR PILE 47
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LENGTH LESS THAN 5T2 FOR PILE 48
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LENGTH LESS THAN 5T2 FOR PILE 49
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LENGTH LESS THAN 5T2 FOR PILE 50
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LENGTH LESS THAN 5T2 FOR PILE 51
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LENGTH LESS THAN 5T2 FOR PILE 52
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LENGTH LESS THAN 5T2 FOR PILE 53
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LENGTH LESS THAN 5T2 FOR PILE 54
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LENGTH LESS THAN 5T2 FOR PILE 55
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LENGTH LESS THAN 5T2 FOR PILE 56
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PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	-18.00	2.00	.00	V	.00	20.00	F
2	-12.00	2.00	.00	V	.00	20.00	F
3	-6.00	2.00	.00	V	.00	20.00	F
4	.00	2.00	.00	V	.00	20.00	F
5	6.00	2.00	.00	V	.00	20.00	F
6	12.00	2.00	.00	V	.00	20.00	F
7	18.00	2.00	.00	V	.00	20.00	F
8	-18.00	6.00	.00	V	.00	20.00	F
9	-12.00	6.00	.00	V	.00	20.00	F
10	-6.00	6.00	.00	V	.00	20.00	F
11	.00	6.00	.00	V	.00	20.00	F
12	6.00	6.00	.00	V	.00	20.00	F
13	12.00	6.00	.00	V	.00	20.00	F
14	18.00	6.00	.00	V	.00	20.00	F
15	-20.50	11.50	.00	V	.00	15.00	F
16	-19.50	11.50	.00	V	.00	15.00	F
17	-18.50	11.50	.00	V	.00	15.00	F
18	-17.50	11.50	.00	V	.00	15.00	F
19	-16.50	11.50	.00	V	.00	15.00	F
20	-15.50	11.50	.00	V	.00	15.00	F
21	-14.50	11.50	.00	V	.00	15.00	F
22	-13.50	11.50	.00	V	.00	15.00	F
23	-12.50	11.50	.00	V	.00	15.00	F
24	-11.50	11.50	.00	V	.00	15.00	F
25	-10.50	11.50	.00	V	.00	15.00	F
26	-9.50	11.50	.00	V	.00	15.00	F
27	-8.50	11.50	.00	V	.00	15.00	F
28	-7.50	11.50	.00	V	.00	15.00	F
29	-6.50	11.50	.00	V	.00	15.00	F
30	-5.50	11.50	.00	V	.00	15.00	F

31	-4.50	11.50	.00	V	.00	15.00	F
32	-3.50	11.50	.00	V	.00	15.00	F
33	-2.50	11.50	.00	V	.00	15.00	F
34	-1.50	11.50	.00	V	.00	15.00	F
35	-.50	11.50	.00	V	.00	15.00	F
36	.50	11.50	.00	V	.00	15.00	F
37	1.50	11.50	.00	V	.00	15.00	F
38	2.50	11.50	.00	V	.00	15.00	F
39	3.50	11.50	.00	V	.00	15.00	F
40	4.50	11.50	.00	V	.00	15.00	F
41	5.50	11.50	.00	V	.00	15.00	F
42	6.50	11.50	.00	V	.00	15.00	F
43	7.50	11.50	.00	V	.00	15.00	F
44	8.50	11.50	.00	V	.00	15.00	F
45	9.50	11.50	.00	V	.00	15.00	F
46	10.50	11.50	.00	V	.00	15.00	F
47	11.50	11.50	.00	V	.00	15.00	F
48	12.50	11.50	.00	V	.00	15.00	F
49	13.50	11.50	.00	V	.00	15.00	F
50	14.50	11.50	.00	V	.00	15.00	F
51	15.50	11.50	.00	V	.00	15.00	F
52	16.50	11.50	.00	V	.00	15.00	F
53	17.50	11.50	.00	V	.00	15.00	F
54	18.50	11.50	.00	V	.00	15.00	F
55	19.50	11.50	.00	V	.00	15.00	F
56	20.50	11.50	.00	V	.00	15.00	F

						910.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-213.0	281.1	937.4	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.61279E+04	.00000E+00	.00000E+00	.00000E+00	.23837E+06	-.65881E+06
.00000E+00	.55780E+04	.00000E+00	-.20972E+06	.00000E+00	-.50932E-10
.00000E+00	.00000E+00	.29651E+06	.32402E+08	.00000E+00	.00000E+00
.00000E+00	-.20972E+06	.32402E+08	.41301E+10	.00000E+00	-.34925E-09
.23837E+06	.00000E+00	.00000E+00	.00000E+00	.62484E+10	-.24561E+08
-.65881E+06	-.50932E-10	.00000E+00	-.34925E-09	-.24561E+08	.20028E+09

56 PILES   1 LOAD CASES

LOAD CASE   1. NUMBER OF FAILURES = 56. NUMBER OF PILES IN TENSION = 42.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	-.1704E-17	-.3997E-01	.6112E-02	-.4726E-04	.2708E-23	-.1585E-19

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE -   1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
2	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*

3	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
4	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
5	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
6	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
7	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
8	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
9	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
10	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
11	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
12	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
13	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
14	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
15	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
16	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
17	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
18	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
19	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
20	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
21	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
22	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
23	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
24	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
25	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
26	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
27	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
28	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
29	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
30	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
31	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
32	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
33	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
34	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
35	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
36	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
37	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
38	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
39	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
40	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
41	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
42	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
43	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
44	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
45	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
46	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
47	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
48	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
49	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
50	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
51	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
52	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
53	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
54	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
55	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
56	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*

****

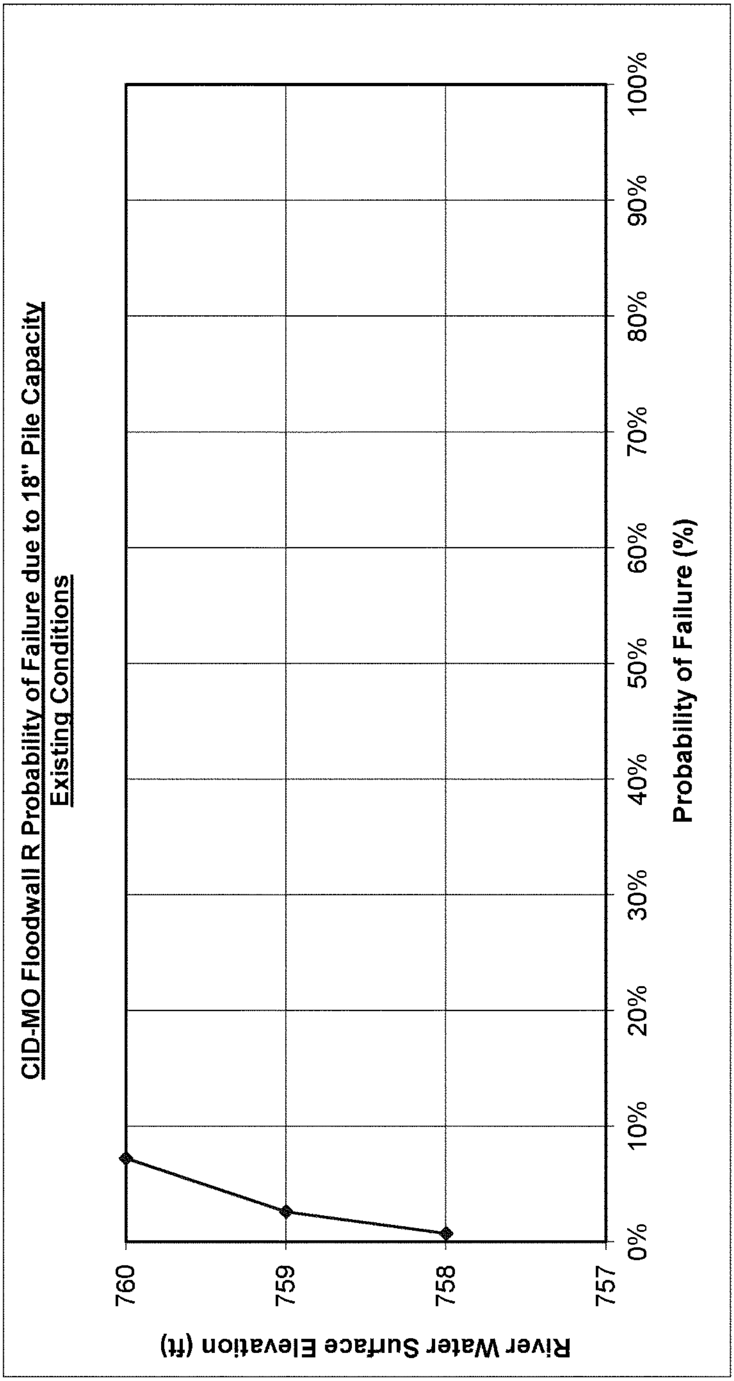
PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-5.6	33.6	241.9	.0	.0
2	.0	-5.6	33.6	241.9	.0	.0
3	.0	-5.6	33.6	241.9	.0	.0
4	.0	-5.6	33.6	241.9	.0	.0
5	.0	-5.6	33.6	241.9	.0	.0
6	.0	-5.6	33.6	241.9	.0	.0
7	.0	-5.6	33.6	241.9	.0	.0
8	.0	-5.6	18.3	241.9	.0	.0
9	.0	-5.6	18.3	241.9	.0	.0
10	.0	-5.6	18.3	241.9	.0	.0
11	.0	-5.6	18.3	241.9	.0	.0
12	.0	-5.6	18.3	241.9	.0	.0
13	.0	-5.6	18.3	241.9	.0	.0
14	.0	-5.6	18.3	241.9	.0	.0
15	.0	-3.2	-2.0	104.4	.0	.0
16	.0	-3.2	-2.0	104.4	.0	.0
17	.0	-3.2	-2.0	104.4	.0	.0
18	.0	-3.2	-2.0	104.4	.0	.0
19	.0	-3.2	-2.0	104.4	.0	.0
20	.0	-3.2	-2.0	104.4	.0	.0
21	.0	-3.2	-2.0	104.4	.0	.0

22	.0	-3.2	-2.0	104.4	.0	.0
23	.0	-3.2	-2.0	104.4	.0	.0
24	.0	-3.2	-2.0	104.4	.0	.0
25	.0	-3.2	-2.0	104.4	.0	.0
26	.0	-3.2	-2.0	104.4	.0	.0
27	.0	-3.2	-2.0	104.4	.0	.0
28	.0	-3.2	-2.0	104.4	.0	.0
29	.0	-3.2	-2.0	104.4	.0	.0
30	.0	-3.2	-2.0	104.4	.0	.0
31	.0	-3.2	-2.0	104.4	.0	.0
32	.0	-3.2	-2.0	104.4	.0	.0
33	.0	-3.2	-2.0	104.4	.0	.0
34	.0	-3.2	-2.0	104.4	.0	.0
35	.0	-3.2	-2.0	104.4	.0	.0
36	.0	-3.2	-2.0	104.4	.0	.0
37	.0	-3.2	-2.0	104.4	.0	.0
38	.0	-3.2	-2.0	104.4	.0	.0
39	.0	-3.2	-2.0	104.4	.0	.0
40	.0	-3.2	-2.0	104.4	.0	.0
41	.0	-3.2	-2.0	104.4	.0	.0
42	.0	-3.2	-2.0	104.4	.0	.0
43	.0	-3.2	-2.0	104.4	.0	.0
44	.0	-3.2	-2.0	104.4	.0	.0
45	.0	-3.2	-2.0	104.4	.0	.0
46	.0	-3.2	-2.0	104.4	.0	.0
47	.0	-3.2	-2.0	104.4	.0	.0
48	.0	-3.2	-2.0	104.4	.0	.0
49	.0	-3.2	-2.0	104.4	.0	.0
50	.0	-3.2	-2.0	104.4	.0	.0
51	.0	-3.2	-2.0	104.4	.0	.0
52	.0	-3.2	-2.0	104.4	.0	.0
53	.0	-3.2	-2.0	104.4	.0	.0
54	.0	-3.2	-2.0	104.4	.0	.0
55	.0	-3.2	-2.0	104.4	.0	.0
56	.0	-3.2	-2.0	104.4	.0	.0

NO FILES WERE GENERATED DURING THIS RUN.
Stop - Program terminated.


CID-MO Floodwall R Probability of Failure due to 18" Pile Capacity
Existing Conditions
River Water Surface Elevation (ft)
760
759
758
757
Probability of Failure (%)
0%
10%
20%
30%
40%
50%
60%
70%
80%
90%
100%

FOR CONTINUATION OF HOUSE DOCUMENT 114-138

KANSAS CITYS, MISSOURI AND KANSAS FLOOD RISK MANAGEMENT PROJECT

SEE PART 2